MSC
MODERN STEEL CONSTRUCTION
June 2009

Riding the Waves
Seismic Upgrades

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Seismic Design
Bolts
Moment Connections

ARE YOU SPECIFYING CUSTOM CONFIGURATION JOISTS?

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3. Provide diagrams showing depths, lengths, pitches, radiuses, slopes, and/or elevations as required to define the joists profiles. CMC Joist & Deck will determine the web configuration for optimum material usage and labor costs.
4. Provide all loads, load categories, load combinations, and deflection requirements.
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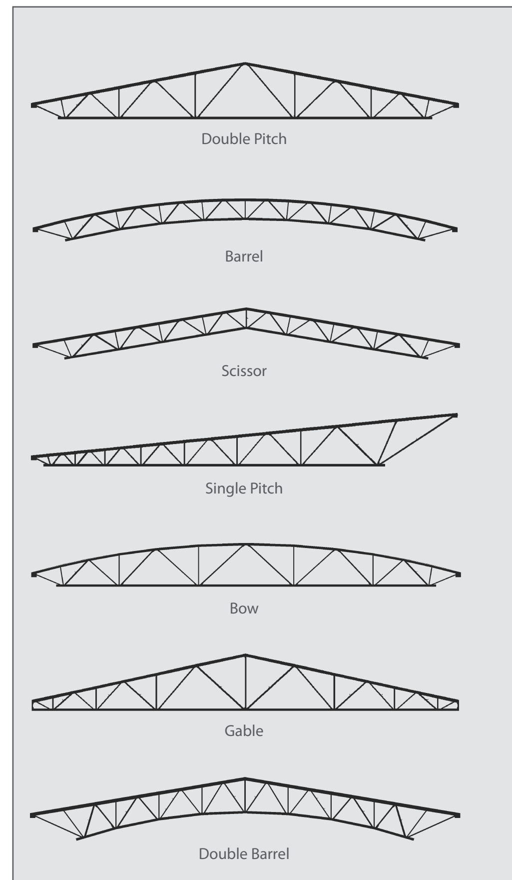

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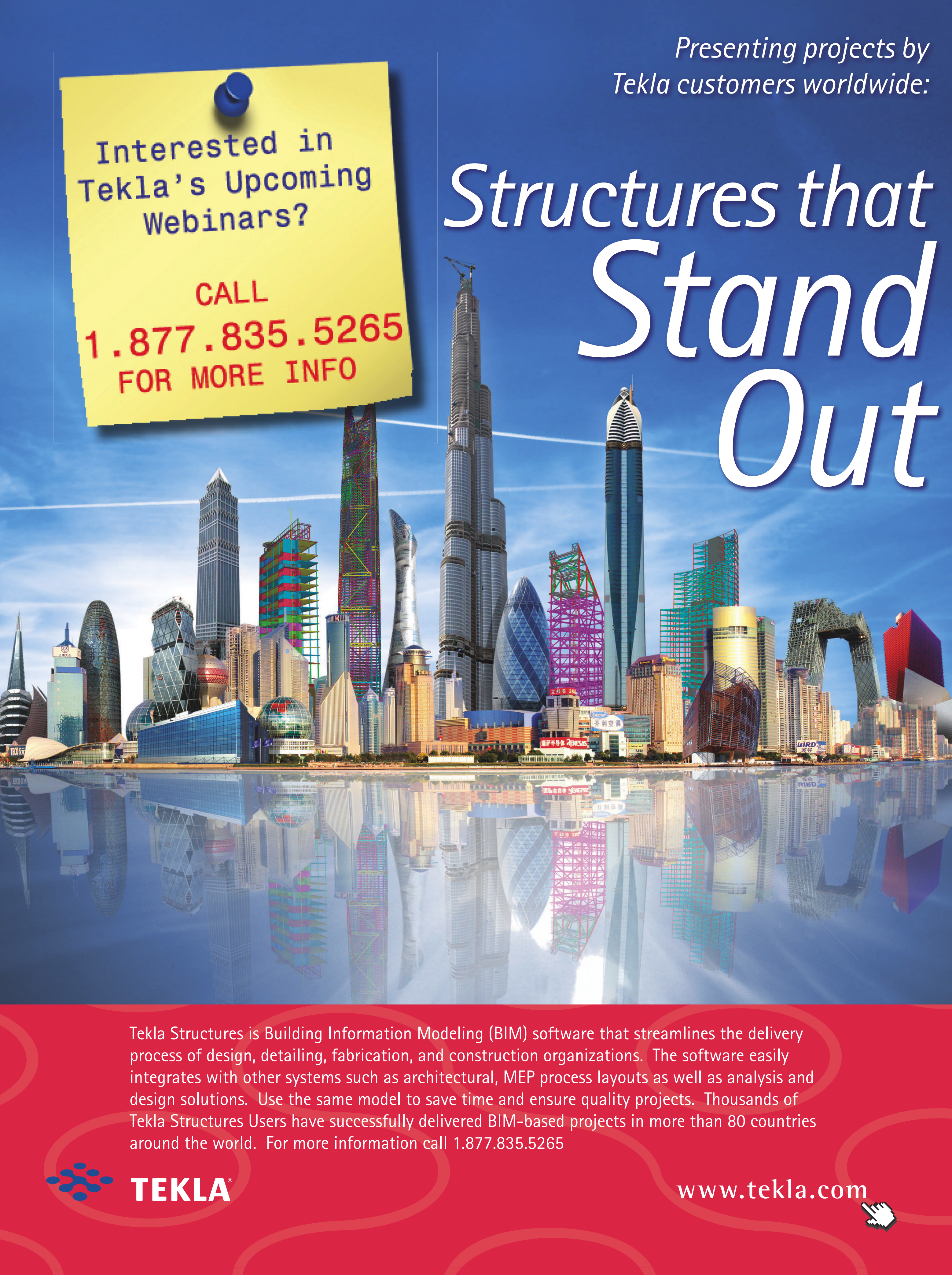
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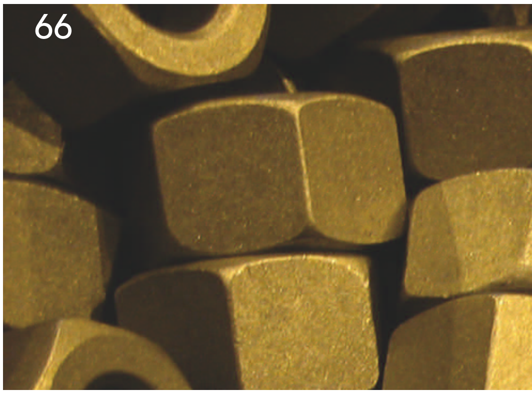
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ON THE COVER: The Tabernacle in Salt Lake City. Photo: Courtesy of Reaveley Engineers + Associates

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editor's note

I LOVE WHEN I GET TO HEAR A REALLY REMARKABLE SPEAKER. Sometimes the speaker is simply awe-inspiring, such as Gene Krantz, the mission control specialist on Apollo 13, who keynoted the Steel Conference back in 2006. Sometimes the talk is inspirational, such as Duane Miller's talk last year on "Important Lessons They Didn't Teach Me At College" (which you can view online at **www.aisc.org/2008nascconline**).

And sometimes the speaker has a simple message but with the potential for a tremendous impact. At the recent Structures Congress (presented by the Structural Engineering Institute), Professor John Breen from the University of Texas at Austin compared the current building codes to the old "Code of the West." Besides being both entertaining and engaging, Breen had an important message: The language used to express today's building codes and specifications is too complex.

Note that Breen wasn't criticizing the content of the current specifications; rather, he was criticizing how they were written.

Specifically, his contention was that some codes and specifications are written in language that only a post-doc could love. To support his argument, he cited the Gunning fog index, a test designed to measure the readability of a sample of English writing. The test, which was developed by businessman Robert Gunning in 1952, posits that a fog index of around 12 approximates the reading level of a high school senior.

The index is calculated with the following algorithm:

1. Take a sample passage of at least 100 words.
2. Find the average sentence length.
3. Count the number of words with three or more syllables (complex words); don't include compound words (such as butterfly) or proper nouns.
4. Add the average sentence length to the percentage of complex works.
5. Multiply by 0.4.

(Of course, today it's easier to visit Google, type in "Gunning fog index calculator," and find a website that lets you paste in a block of text and which then calculates the fog index. For example, according to one of these online calculators, this editorial has a fog index of 10.14 – which is just about perfect for readability by an educated audience.)

According to Breen (and what added impact to his talk was that he was a major contributor to both ASCE 7 and ACI 318), some structural codes have a fog index as high as 29!

Will Breen's keynote talk have an impact on future codes? Judging by the buzz it generated both at the conference and afterwards, I think it will. And if it does, future generations of designers and code officials will know who to thank.

Scott Melnick

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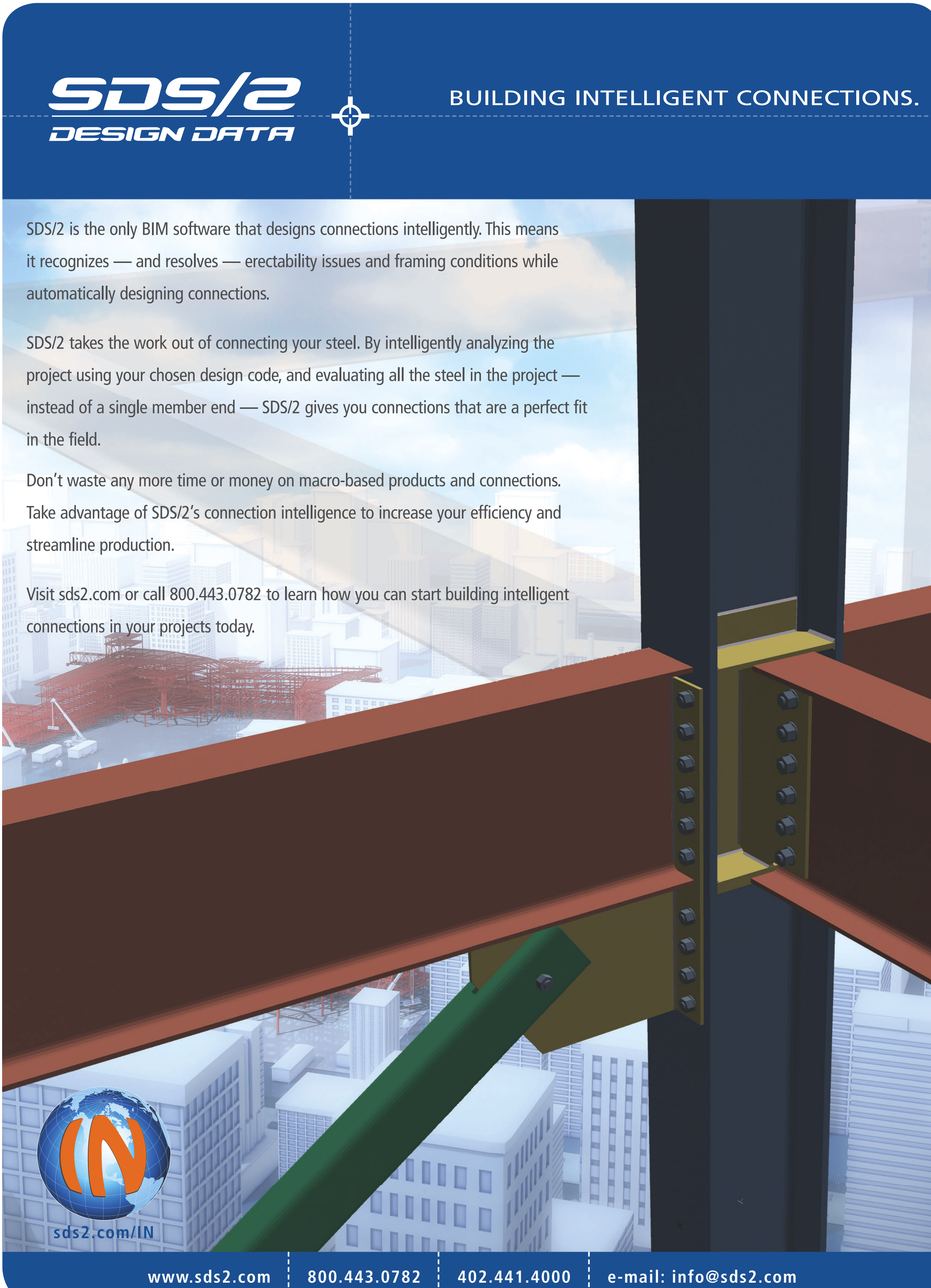
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IF YOU'VE EVER ASKED YOURSELF "WHY?" about something related to structural steel design or construction, *Modern Steel Construction*'s monthly Steel Interchange column is for you! Send your questions or comments to solutions@aisc.org.

Galvanized Slip-Critical Connections

Section 7.2 of the AISC *Seismic Provisions* indicates that bolted joints must have Class A faying surfaces. Section 3.2.2(c) of the RCSC *Specification for Structural Joints using ASTM A325 or A490 Bolts* indicates that galvanized faying surfaces are designated as Class C. Does this mean that we are unable to use steel members that are galvanized in the vicinity of the connections, for high-seismic applications?

The classes of faying surface finish requirements have been revised in the 2005 AISC *Specification*, now only including Class A and Class B requirements. The 2004 RCSC *Specification* was based on the three Class distinctions. The Commentary to Section J3.8 (page 349) of the 2005 AISC *Specification* discusses this revision. The previous Class A and Class C categories have now been consolidated into one Class A, which includes hot-dip galvanized and roughened surfaces.

Kurt Gustafson, S.E., P.E.

Floor Plate

A note at the bottom of the Floor Plate Bending Capacity table on p. 2-145 in the 9th edition ASD *Manual* indicates that the loads are based on an extreme fiber stress of 16 ksi and simple-span bending. The 16 ksi allowable stress seems to be very conservative, assuming that the plates would likely have a yield strength of not less than that for A36 steel. What is the 16 ksi allowable based on?

You are right that the tables published in the *Manual* for simple-span flexure of floor plates may be conservative. However, these tables are merely design aides based on the conservative assumptions that are stated. Floor plate is commonly specified as ASTM A786, which is generally a commercial grade steel with no defined strength level, and this table allows for a very low strength level product. The responsible design professional always has the option of making their own analysis based on known parameters of the material they are working with, rather than use what they may deem to be conservative design aides. However, floor plate design is usually controlled by deflection anyway.

Kurt Gustafson, S.E., P.E.

Rotational Restraint at Support

AISC *Specification* Section J10.7 requires full-depth stiffeners at the "unframed ends of beams and girders." What does this mean? Would an example be a girder bearing on a column with no beam framing into it at the column?

Yes, this section addresses situations such as the end of a beam that bears on column cap plate. Unless the column top is restrained, the beam might twist or the web might distort, allowing the bottom flange to move transversely. This creates a dangerous situation, because the column below was designed assuming a pinned-pinned condition with its top is restrained against lateral displacement. If a brace is provided to restrain the top of the column, the beam end is framed. If not, stiffeners can be used as required in Section J10.7. Note that the concern for column stability also exists when girders frame continuously over the top of the column. See Section 2 of the 13th edition AISC *Steel Construction Manual* for further information.

Brad Davis, Ph.D., S.E.

Bolting for High-Seismic Applications

Are slip-critical connections required for seismic connections? And if so, for what seismic design category are they necessary?

In high-seismic applications, slip resistance is required, but the connections are designed for bearing values. According to Section 7.2 of the 2005 AISC *Seismic Provisions* (AISC 341) (a free download at **www.aisc.org/2005seismic**), "All bolts shall be pretensioned high-strength bolts and shall meet the requirements for slip-critical faying surfaces … with a Class A surface." Also, "The available shear strength of bolted joints using standard holes shall be calculated as that for bearing-type joints…" This is applicable for high-seismic applications where the requirements in AISC 341 must be met.

Amanuel Gebremeskel, P.E

Nut Engagement

We have a situation where bolts have been installed too short (the bolt tip is below the top of nut) in a steel-to-steel joint. Is there a way to assess the reduced capacity based on the percentage of thread engaged?

The 2004 RCSC *Specification* (a free download at **www.boltcouncil.org**) requires that "the bolt length used shall be such that the end of the bolt extends beyond or is at least flush with the outer face of the nut when properly installed." Thus when the bolts are "short," the installation is non-compliant. There is no reduced value permitted by the specification. The bolts should be replaced with bolts of the correct length.

Kurt Gustafson, S.E., P.E.

Hole Sizes for Galvanized Bolts

An engineer designed the structural steel connections using standard holes in all plys for ASTM A325-N bolts such that the connections need not consider slip-critical limit states. The steel is to be hot-dip galvanized. The galvanizer is requesting that the standard holes be increased by an additional tolerance of 1⁄16 in. to account for the coating thickness. I'm hesitant to grant approval for a hole size that would require slip-critical limit states to dictate connection design. If the hole size is increased, would the connection design need to be reevaluated for slip-critical conditions?

Increasing the hole size to account for the galvanizing in a bearing condition is not an accepted practice and is not allowed by the AISC *Specification* or the RCSC *Specification*. If the holes are oversized the connection must be designed as slip-critical.

Larry S. Muir, P.E.

steel interchange

Stiffeners for an EBF Link

Commentary Section C15.3 in the AISC *Seismic Provisions* indicates that for EBF links that are less than 25 in. deep, the stiffener need be on one side only. What is the interpretation of "need be?" Does it mean "must be" or "may be?" Many practicing engineers are interpreting this as "may be." When EBFs were tested, what was the protocol? Have they tested intermediate stiffeners on one side only? Is there any detrimental effect on the inenelastic rotation of link beam due to increased rigidity of link beam when stiffeners are used on both sides?

Use of either one-sided or two-sided intermediate stiffeners is allowed for beams up to the indicated depth. Both one-sided and two-sided specimens have been tested, with similar loading protocols to what is used for moment connections (increasing the displacement incrementally until failure). The drift limits in the AISC *Seismic Provisions* were based on these tests. No difference in performance was noted between one-sided and two-sided specimens. The key item is that the stiffener must be stiff enough to force the link web to buckle in the panels between stiffeners, rather than over length of the link. The thickness requirements for stiffeners in the AISC *Seismic Provisions* are adequate to make this happen for the single-sided cases.

James O. Malley, Senior Principal
Degenkolb Engineers

Tensile Strength of Anchor Rods

Where does one find values for f_{uta} (specified tensile strength of anchor steel) as used in Appendix D of ACI 318? Also, is the f_{ya} (specified yield strength of anchor steel) equal to F_{nt} given in Table J3.2 of the 2005 AISC *Specification*?

Generally, the required minimum tensile stress for the material type can be found in the applicable ASTM Standard. The value of F_u for many types of ASTM materials used for anchor rods covered by the AISC *Specification* also are listed in Table 2-5 of the 13th edition AISC *Steel Construction Manual*.

The answer is "no" to the second question. The values of F_{nt} in Table J3.2 in the 2005 AISC *Specification* provide the nominal tensile stress for use with ϕ or Ω in design, and are adjusted to account for the difference between nominal bolt body area and threaded area. This is not the yield strength. Some anchor rod materials have a defined yield point, while others do not. The tensile stress as used in the ACI 318 Appendix D approach is based on the tensile stress area at the thread. Therefore, one needs to be careful when comparing the two approaches to anchor rod evaluation.

Kurt Gustafson, S.E., P.E.

Punching Shear

Why is there a requirement to check punching shear on the wall of an HSS column with a single-plate shear connection, but no similar check when connecting to the web of a W-Shape?

Punching shear can occur at a W-shape column or girder web. However, it is not usually a consideration, because W-shape column and girder webs are usually thick enough that punching shear won't control. Using Equation K1-10 of the 2005 AISC *Specification*, for a ⅜-in. ASTM A36 shear tab and an ASTM A992 web, the web would have to be less than 0.208 in. thick for punching shear to control. It would be very unusual for a W-shape column to have a web thickness less than this.

There are three W-shape beam sections that have a web thickness less than 0.208 in. These are rarely, if ever, used as girders. Even if one of these sections (W8×10, W10×12, or W12×14) is used as a girder, it will not have adequate torsional stiffness to allow punching shear to be a realistic limit state. If a shear tab is on both sides of the web, then the supporting member won't rotate much, but punching shear also may not occur in that case, because shear tab on the opposite side will be trying to rotate in the opposite direction. (Note that a single-sided shear tab is always the case for an HSS column.)

So, punching shear is possible for a web supporting a shear tab, but it is only realistic for HSS columns, because those columns provide significant rotational restraint and some of those sections have extremely thin walls.

Brad Davis, Ph.D., S.E.

Shear Lag Factor

Table D3.1 of the 2005 AISC *Specification* gives shear lag factors (U) for various cases of tension connections. I have a situation like Case 4, where two plates are transmitting tension through longitudinal welds only. The U-factors are based upon the length of the weld (l) and the width of the plate (w). No U-factors are tabulated for the condition where $l < w$. The plate I have is 4 in. wide and the weld can only be 2 in. long. What U-factor is appropriate for this situation?

Such a weld connection configuration does not meet the AISC *Specification* requirement as stated in Section J2.2b: "If longitudinal fillet welds are used alone in end connections of flat-bar tension members, the length of each fillet weld shall not be less than the perpendicular distance between them." Thus there is no U-factor listed as appropriate for this detail because it represents a condition in which the shear lag effect is likely to cause rupture to occur in a manner that is not well predicted by the methods we use in design.

Kurt Gustafson, S.E., P.E.

The complete collection of Steel Interchange questions and answers is available online. Find questions and answers related to just about any topic by using our full-text search capability. Visit Steel Interchange online at **www.modernsteel.com**.

Kurt Gustafson is the director of technical assistance and Amanuel Gebremeskel is a senior engineer in AISC's Steel Solutions Center. Larry Muir and Brad Davis are part-time consultants to AISC. James O. Malley is Senior Principal at Degenkolb Engineers.

Steel Interchange is a forum to exchange useful and practical professional ideas and information on all phases of steel building and bridge construction. Opinions and suggestions are welcome on any subject covered in this magazine.

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If you have a question or problem that your fellow readers might help you solve, please forward it to us. At the same time, feel free to respond to any of the questions that you have read here. Contact Steel Interchange via AISC's Steel Solutions Center:

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steel quiz

LOOKING FOR A CHALLENGE? *Modern Steel Construction*'s monthly Steel Quiz tests your knowledge of steel design and construction. Most answers can be found in the 2005 *Specification for Structural Steel Buildings*, available as a free download from AISC's web site, **www.aisc.org/2005spec**. Where appropriate, other industry standards are also referenced.

Questions 1 through 5 relate to the 2005 AISC *Code of Standard Practice*.

1 True/False: When there is a discrepancy between the design drawings and specifications, the specification governs.

2 What is the camber tolerance for a beam that is 30 ft long?
(a) +/- ¼ in.
(b) -0 / + ½ in.
(c) -0 / + ¾ in.
(d) None of the above

3 Is the fabricator responsible for supplying steel stud shear connectors to the field?

4 True/False: Lintels are considered structural steel in Section 2 of the 2005 AISC *Code of Standard Practice*.

5 What is the permissible variation in distance between the centers of anchor rod groups?
(a) 1/16 in.
(b) ⅛ in.
(c) ¼ in.
(d) ½ in.

6 When a single-plate shear connection is welded to the face of an HSS wall, how can the designer preclude punching shear of the HSS wall?

7 Is the reuse of high-strength bolts permitted by the AISC *Specification*?

8 Is load rating by testing of existing floor or roof systems permitted by AISC?

9 Does AISC provide any guidance for load transfer and stiffening requirements when framing a beam to the weak axis of a column?

10 True/False: Equations H1-1a and H1-1b in the 2005 AISC *Specification* can be used to design WT shapes under combined compression and flexural loading.

TURN TO PAGE 14 FOR ANSWERS

steel quiz

ANSWERS

1 False. According to Section 3.3 of the 2005 AISC *Code of Standard Practice*, when there is a discrepancy between the design drawings and specifications, the design drawings govern. Two additional notes: (1) It is not the contractor's responsibility to find discrepancies, but (2) if discovered before work is performed, they are required to be reported for resolution.

2 (b) According to Section 6.4 of the 2005 AISC *Code of Standard Practice*, the camber tolerance for members of 50 ft or less in length is -0 / + ½ in.

3 According to Section 7.8.3 of the 2005 AISC *Code of Standard Practice*, steel stud shear connectors are supplied by the fabricator if they are attached to structural steel in the shop. Otherwise, the responsibility for providing steel stud shear connectors must be specified in the contract documents. Note that OSHA requirements do not allow shop attachment of shear studs and similar items in the shop when they would create a tripping hazard in the field.

4 Trick question. The answer is true for lintels that are attached to the structural steel frame. Otherwise, the answer is false.

5 (b) According to Section 7.5 of the 2005 AISC *Code of Standard Practice*, the variation in distance between centers of anchor rods in groups should not exceed ⅛ in.

6 Punching shear of an HSS wall that has a single-plate shear connection attached to it can be precluded by ensuring that the force from the plate does not exceed the shear strength of the wall. This is commonly done using the inequality provided in Equation K1-10 of the 2005 AISC *Specification*.

7 The AISC *Specification* incorporates the reuse provisions in the RCSC *Specification* by reference. The RCSC *Specification* permits reuse of black ASTM A325 bolts, but not ASTM A490 bolts or galvanized ASTM A325 bolts. This is also discussed in AISC Design Guide 17 (**www.aisc.org/epubs**).

8 Yes. The provisions of Appendix 5.4 in the 2005 AISC *Specification* permit the testing of existing floor and roof structures to establish a load rating.

9 Yes. Part 10 of the 13th edition AISC *Manual* (page 10-140) and information in AISC Design Guide No. 13 (Appendix B) provide guidance on the design and details for beams that frame to the weak axis of a column.

10 False. The WT shapes shown in Part 1 of the 13th edition AISC *Manual* do not satisfy the I_{yc}/I_y ratio limits set in section H1.1. Therefore, Equation H2-1 is used for WT design with combined loading.

Anyone is welcome to submit questions and answers for Steel Quiz. If you are interested in submitting one question or an entire quiz, contact AISC's Steel Solutions Center at 866.ASK.AISC or at **solutions@aisc.org**.

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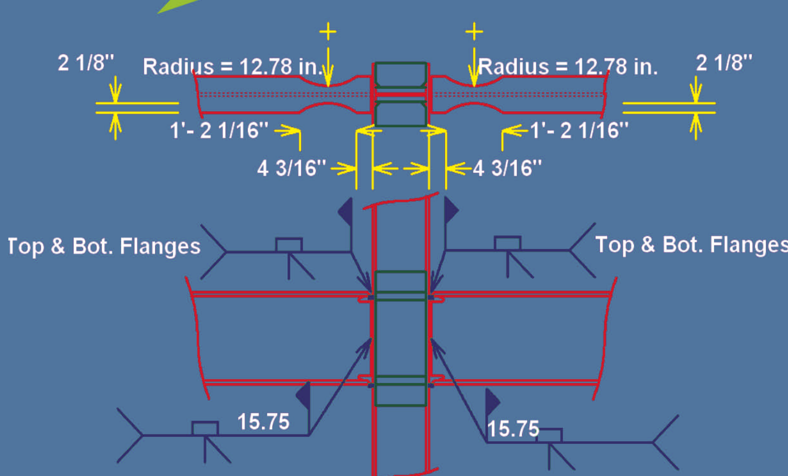

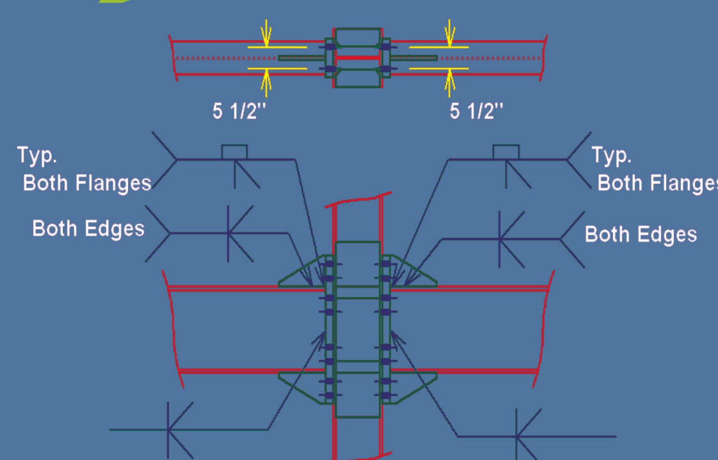

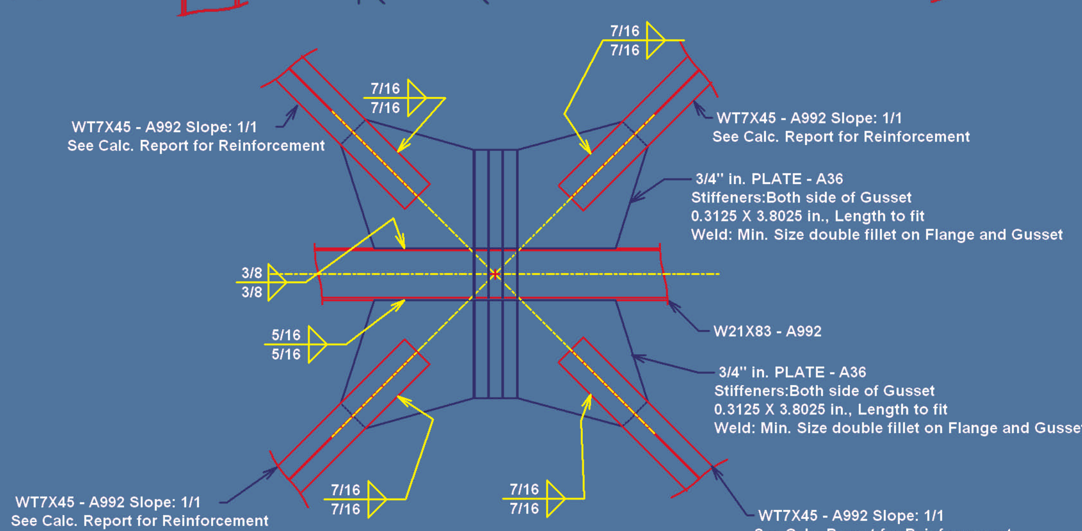

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AWARDS

Call for Entries for AWS 7th Annual Image of Welding Awards

Jessica Sladek

The American Welding Society (AWS) has issued a call for entries for the 7th Annual Image of Welding Awards program, which salutes the year's most outstanding public initiatives and programs that promote the image of welding.

The program is open to all welding industry professionals, and the awards are issued in six categories. All entrants, organizations, and groups may be nominated for multiple categories, and self-nominations are also welcome. Winners will be honored at the Image of Welding Awards Ceremony to be held during the FABTECH International and AWS Welding Show on November 15–18, 2009 at McCormick Place in Chicago.

The Image of Welding Awards Program recognizes outstanding achievement in the following categories:

- individual (you or other individual)
- section (AWS local chapter)
- small business (less than 200 employees)
- large business (200 or more employees)
- distributor (welding products)
- educator (welding teacher at an institution, facility, etc.)
- educational facility (any organization that conducts welding education or training)

Nominations will be judged by the Welding Equipment Manufacturers Committee (WEMCO), a standing committee at AWS—composed of executives of welding industry suppliers—that promotes the welding equipment market.

To see past Image of Welding Award winners and to submit a nomination, download the PDF nomination form online at **www.aws.org/awards/image.html**. You can return your completed form to **image@aws.org** or 305.443.1552 (fax). For questions and other requests, please call 800.443.9353. The entry deadline is July 10, 2009.

Innovative Joist Design

Have an innovative joist project? The Steel Joist Institute is now accepting entries for its 2009 Design Awards. The winning entries will be announced in November 2009 and the company with the winning project in each category will be awarded a $2,000 scholarship in its name to a school of its choice for an engineering student.

The awards will be presented in three categories:

- industrial (distribution centers, warehouses, and light manufacturing)
- non-industrial (office buildings, schools, and churches)
- unique applications (projects with a unique application of steel joists)

Eligible projects—new or major retrofits/expansions—must be located in the United States, Canada, or Mexico, and the steel joists and/or joist girders must be manufactured by an active member of the Steel Joist Institute. (A list of members can be found here: **www.steeljoist.org/members**.) Projects must have been constructed within the last three years.

Companies can submit more than one project, and each project will be judged separately. Judging of each project is evaluated and based on flexibility, speed of construction, value, and aesthetic considerations.

Visit **www.steeljoist.org/awards** by July 24, 2009 for an online entry form and a complete listing of rules.

And to see the winners of AISC's own awards program, the IDEAS² Awards, visit **www.modernsteel.com/2009IDEAS**.

STANDARDS

Mechanical Tests, Metric Standards

A new ASTM International standard will serve as a guide for manufacturers and laboratories that make and test steel products according to standards using the SI system of units.

The new standard, ASTM A1058, *Test Methods for Mechanical Testing of Steel Products—Metric*, arose from a need for a stand-alone metric steel testing standard, according to Lester Burgess, director of quality with TSP/U.S. Bolt Manufacturing and chair of Subcommittee A01.13 on Mechanical and Chemical Testing and Processing Methods of Steel Products and Processes.

The new standard follows a distinctly different format from that of well-known testing standard ASTM A370, *Test Methods and Definitions for Mechanical Testing of Steel Products*. For example, ASTM A1058 does not include the product annexes found in ASTM A370. ASTM A1058 provides detailed direction for mechanical testing and includes coverage of international standards. The new standard references and cross-references international standards such as the European Committee for Standardization, the International Organization for Standardization, and the Japanese Standards Association.

ASTM A1058 was developed by a task group under the direction of Subcommittee A01.13, part of ASTM International Committee A01 on Steel, Stainless Steel, and Related Alloys. Committee A01 subcommittees will begin referencing the new standard as it applies to their individual specifications.

ASTM International standards are available for purchase at **www.astm.org**.

PUBLICATIONS

Public Review of 2010 AISC Seismic Provisions

The 2010 draft of the AISC *Seismic Provisions for Structural Steel Buildings* is available for public review from May 1 to June 15, 2009. The *Provisions* are available for download from the AISC web site at **www.aisc.org/AISC341PR1**, along with the review form, during this time. A summary of some of the major revisions is included with the review form. Copies of the draft *Provisions* are also available (for a $12 nominal charge) by calling 312.670.5411.

Please submit comments using the form provided online to Cynthia J. Duncan, AISC's director of engineering, at **duncan@aisc.org** by June 15, 2009 for consideration.

EVENTS

South Africa to Host Mining Conference

The Southern Africa Institute of Steel Construction will host an international conference, Structures for Mining and Related Materials Handling, in Sun City, South Africa November 9–12, 2009. The conference will be aimed at structural and mechanical engineers responsible for the design, construction, and maintenance of structures associated with mining. Presentations will deal with the variety of structures required for extracting mineral resources from the ground and conveying, storing, and processing the materials. Additional information is available at **www.saisc.co.za**.

Academy Prepares Women for Leadership Roles

The National Center for Construction Education and Research (NCCER) and the National Association of Women in Construction (NAWC) will host the third annual Women's Leadership Academy on June 27–30, 2009 in Florissant, Colo.

The Women's Leadership Academy targets business owners, education directors, office managers, training coordinators, or anyone interested in learning how to be a more effective leader. It consists of three days of intense training sessions specifically geared towards women covering such topics as powerful language skills, gender-based power in business leadership styles, negotiating techniques, time management, productivity, and conflict resolution. Participants also receive lasting network opportunities by sharing their experiences with peers from around the nation.

The Women's Leadership Academy is led by quality instructors that motivate and engage participants through a series of team-building exercises, group projects and hands-on activities. Tuition for the academy is $1,495 and includes lodging, all meals, course materials, and airport transportation.

Upon completion of the academy, participants will receive continuing education units and may receive industry-recognized credentials from NCCER's National Registry. To register for the Academy, visit **www.nccer.org/leadership**.

news & events

EVENTS

Domestic Steel Industry to Open its Doors this September

interact. learn. build. STEEL DAY2009 www.SteelDay.org

On September 18, architects, engineers, contractors, and others in the AEC community will have the opportunity to visit steel fabricators, mills, service centers, and other facilities throughout the country. The occasion? SteelDay.

Hosted by AISC, SteelDay 2009 is the first national event dedicated to providing the AEC community with accessibility to the latest happenings in the structural steel industry. To announce SteelDay 2009, AISC recently launched a new web site and portal, **www.steelday.org**, which features information and resources on where all of the action is taking place, including a map of the event locations and how to attend or host an event. The event was also promoted at this year's NASCC in Phoenix (see **www.modernsteel.com/2009NASCC** for coverage of the conference).

Currently, more than 100 SteelDay events are scheduled in 43 states for participants to tour facilities and jobsites, attend educational seminars, network, and see how the structural steel industry is contributing to building America.

"SteelDay is a unique chance for participants to receive hands-on education about the latest advances in the structural steel industry and witness new technologies first-hand," said Chris Moor, AISC's industry mobilization director. "AISC holds tours and seminars throughout the year in specific locations, but we wanted to do something on a grand scale where more people could get these types of learning experiences without having to travel very far."

AISC member Lyman Zolvinski, president of structural engineering consulting firm Zolvinski Engineering in Michigan City, Ind., attended an AISC seminar at a service center last year and hoped to see more fabricator and mill tours become available in his local area. "A wealth of information can be obtained by visiting service centers, fabricators, and mills that can't be put into literature," said Zolvinski. "It also keeps one up to date on the latest supply and fabrication techniques and availability." Zolvinski plans to attend a steel mill tour in Indiana on SteelDay.

During SteelDay, participants will gain hands-on knowledge about structural steel's key benefits and features such as sustainability, availability, speed, and cost. They can also observe how design affects production and efficiency, prompting advances that have resulted in high-performance projects that minimize construction's impact on our planet while also saving time and money.

Visit **www.steelday.org** for more information.

letters

Missed it by that Much

The following errors have been found in our article "Design of Vertical Bracing Connections for High-Seismic Drift" in the March 2009 issue:

1. In Figures 4 and 6, the 19-in.-long fillet welds of the HSS brace to the gusset plate should be $\frac{9}{16}$ in., not $\frac{5}{16}$ in.

2. In Figure 7, the slots should be $1\frac{3}{16} \times 2\frac{1}{2}$, not $1\frac{1}{16} \times 2\frac{1}{2}$.

William A. Thornton and Larry S. Muir

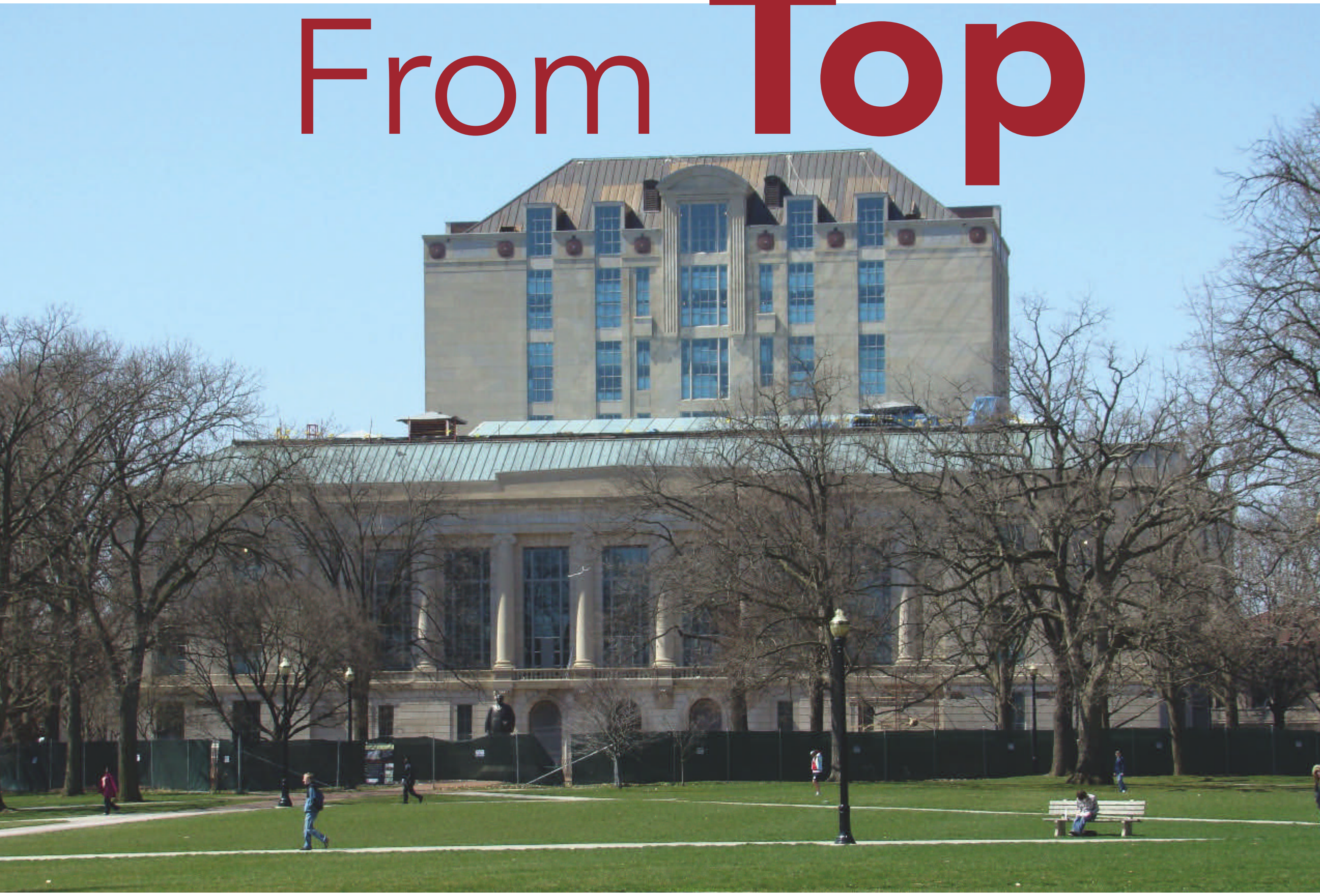
*(A corrected version is posted at **www.modernsteel.com/0309seismic**.)*

Sound Advice

I am an assistant professor at the University of Nevada–Las Vegas School of Architecture, where I teach construction technology to upper-division students. I've just read Rob Kincher's article "The Sounds of Silence" in the April 2009 issue (**www.modernsteel.com/0409silence**). Mr. Kinchler has hit the nail right on the head. The text we have been using is so weak on properly explaining Sound Transmission Class. The article is perfect in explanation and comparisons of various materials and systems.

Jay Barry

From Top



BY STEPHEN METZ, P.E.

to Bottom

Ohio State's main library receives a structural upgrade, from the basement to the attic.

ONE OF THE HALLMARK STRUCTURES for any university or college is the library, and it's not uncommon for the library to be one of the oldest buildings on a campus. Over the years, the weight of time—and books—can take its toll, thus making renovation necessary.

One such library, The Ohio State University's William Oxley Thompson Library, recently underwent a $105 million renovation, including a complete renovation of the existing buildings, partial historic renovation and preservation of the original building, and a four-story addition.

Reviving the Revival

Occupying extremely visible and high-profile real estate on the campus, the library is located on the west end of the historic Oval green space in the middle of campus. Originally constructed in 1913, the library was designed in the Second Italian Renaissance Revival style. The structure of the three-story building was a combination of exterior and interior brick bearing walls and a steel frame. Steel trusses were used to support the roof.

The 11-story stacks tower was added to the west of the original library in 1952 to provide additional study and stack space. The structure of the stacks addition was a combination of structural steel and concrete. The interior columns were composite steel and concrete while the floors are typically a 10-in. two-way concrete slab. Steel channels were welded to the columns to transfer shear from the slab to the columns. A third floor was added in 1966, which divided the reference hall in half. In 1977, because of the continued growth of the university, three more stories were added to the west side of the stacks tower.

All of these various additions created a space that was not unified. Much of the original grandeur of the 1913 structure had been lost and the space had become dark and uninviting. Recognizing that the library was in need of a major transformation to restore its beauty and function, the university embarked on a feasibility study in 2001 followed by the start of design work in 2003. The architect developed a comprehensive plan that would provide an open and unified space and restore some of the historic elements.

Seismic Retrofit

Early in the design process, a seismic analysis of the additions was conducted, revealing a deficiency in the 1952 addition that resulted in the need for a seismic retrofit. The university decided that the lateral load resisting system should be retrofitted to meet the current building code requirements, the 2005 Ohio Building Code. Floors two through ten, with the exception of the mechanical floor at the fifth floor, were programmed to continue being used as book stack space. This resulted in very closely spaced shelving units to provide an efficient use of storage space. The use of conventional braced frames or shear walls would have reduced the number of volumes that could be stored on each floor, due to the size of the members. Hence we opted for steel plate shear walls (SPSWs).

By using the ¼-in.-thick SPSWs, the same number of volumes could be stored on each floor as before the retrofit, which was a tremendous asset to the university. In the end, the SPSW proved to be a cost-effective solution, because the university would have had to build additional, unprogrammed space to make up for the lost shelving.

Removing the Bearing Walls

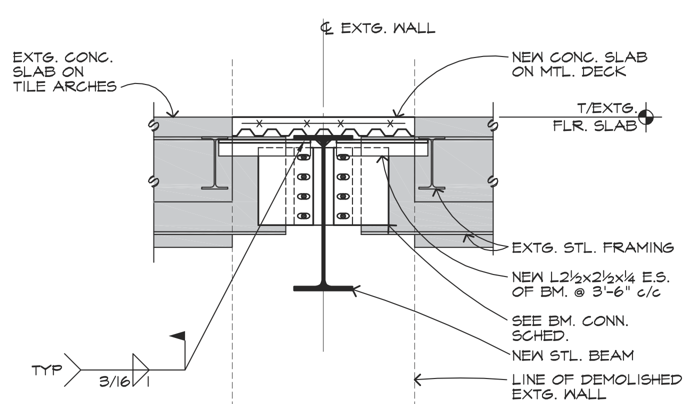
In the 1913 structure, all three of the floors and the basement were partitioned into smaller areas by the interior brick bearing walls, some more than 2 ft thick. The new design of the space called for the basement and first floor to be entirely free of the bearing walls. The walls at the upper floors also had to be removed to allow for the new four story high atrium in the middle of the space. To open the space and allow for the atrium, the bearing walls and large areas of the floors were entirely removed. Temporary shoring was installed during construction to support the floor and roof framing that was left in place. Steel framing was inserted into the building at the locations of the brick bearing walls. The new framing supports the existing floor and roof framing while allowing for the openings at the new atrium.

The north and south walls of the second floor reading room were also brick bearing walls. The historic renovation portion of the project dictated that these walls remained in place so the original plaster details could remain and be restored. However, the brick walls at the first floor and the basement had to be removed. To accomplish this, temporary shoring was erected to support 30-ft-long W24×94 needle beams that were spaced at 4 ft on center. After the walls were removed, double W30×235 beams were placed tight to the underside of the needle beams. The W30 beams were supported on new wide-flange columns. New masonry was added around the needle beams and the beams were cut off on each side of the wall, leaving a piece of the needle beam in the wall.

Constructability was the primary reason why steel framing was selected; it would have been nearly impossible to place concrete in the renovated portions of the building. Further, the selection of steel aided the schedule. Steel was able to be erected faster than concrete could be placed. It also provided the necessary flexibility for attaching to the existing framing. Because of the existing steel members bearing on the brick walls, the exact location of the ends of the members could not be determined until after demolition was complete. By using steel, adjustments were able to be made during construction, based on the actual conditions (see Fig. 1).

Modifications to the existing trusses in the library's attic.

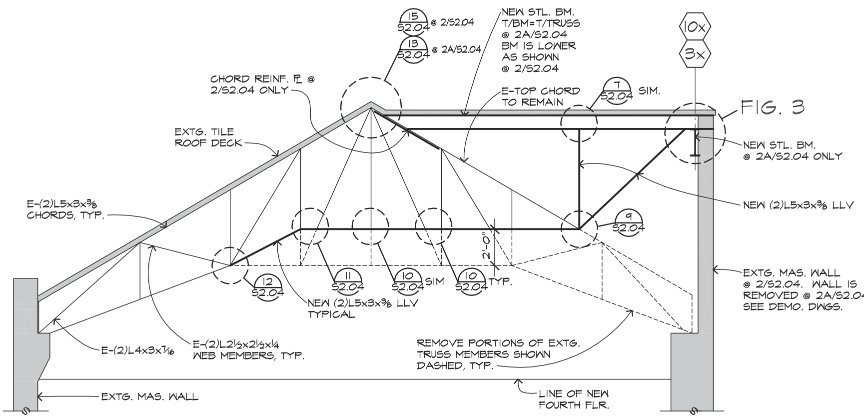
New floor framing at the library's east atrium.



SECTION FIG. 1

¾" = 1'-0"

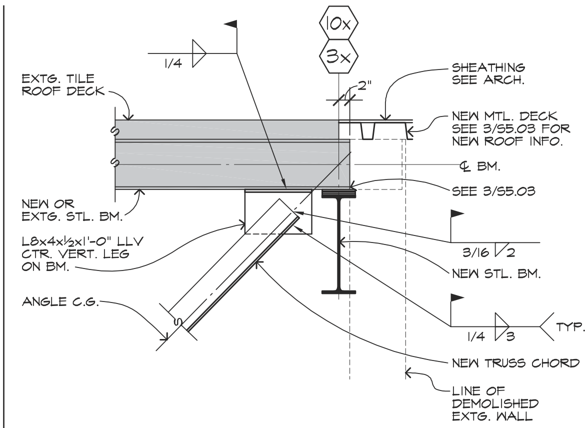
Photos: Courtesy Dave Lee with Acock Associates; Details: Courtesy Shelley Metz Baumann Hawk



TRUSS ELEVATION FIG. 2

¼"=1'-0"

NOTE: ALL NEW MEMBERS SHALL BE COMPLETELY INSTALLED PRIOR TO REMOVAL OF ANY PORTIONS OF THE EXISTING TRUSS



SECTION FIG. 3

¾" = 1'-0"

Cleaning Out the Attic

The original 1913 building had an attic above the third floor. The architect wanted to transform the space to house mechanical equipment, as well as add a fourth floor for office space. In this space, the bearing elevation of the roof trusses was approximately 4 ft above the new fourth floor elevation. The 13-ft truss spacing allowed for the offices to be built such that the office walls aligned with the trusses. However, the corridor outside of the offices would be impossible if the truss bearing elevation was not modified. An additional challenge was that the existing copper roof had to be maintained. To accomplish this, we modified the trusses in place (see Fig. 2). The bottom chords of the trusses had to be reconfigured to allow for the required headroom over the new corridor. The existing trusses were made up of double angles with gusset plate connections. New gusset plates and double-angle members were added before the existing members were removed. Since the end of the truss was bearing on the brick walls that were removed, new connections between the existing truss members and the new steel framing were designed (Fig. 3).

In addition to the original structure, the attic in the stacks tower was also transformed. The attic was originally used for mechanical space, which was programmed to be placed elsewhere in the building. One of the highest points on campus, the space features beautiful views. To capitalize on this, it was decided to transform the space into

Skylight trusses at the east atrium.

Needle beams for temporary support of the reading room walls.

an honors student study area that can also be used for events. Windows and dormers were added to the space to provide views of the Oval. The steel-framed, hipped roof over this space is supported by two interior column lines. The steel framing bears on the attic slab at the perimeter. To provide a more open space for campus events, four interior columns had to be removed. W30×148 girders were added to support the roof framing in the area of the removed columns. To provide openings for the new windows and dormers, portions of the existing roof framing had to be removed and reframed. Curved HSS members and trusses were used to frame the roof over the new dormer areas.

In all, the project includes 340 tons of new steel and is expected to be done in time for the fall academic quarter this year. It demonstrates a complete transformation of space and gives new life to what has always been a focal point of the campus. A dark, enclosed environment has yielded to bright, airy space, not only enhancing the building itself but also the entire experience of going to the library. MSC

Stephen Metz is a principal at Shelley Metz Baumann Hawk.

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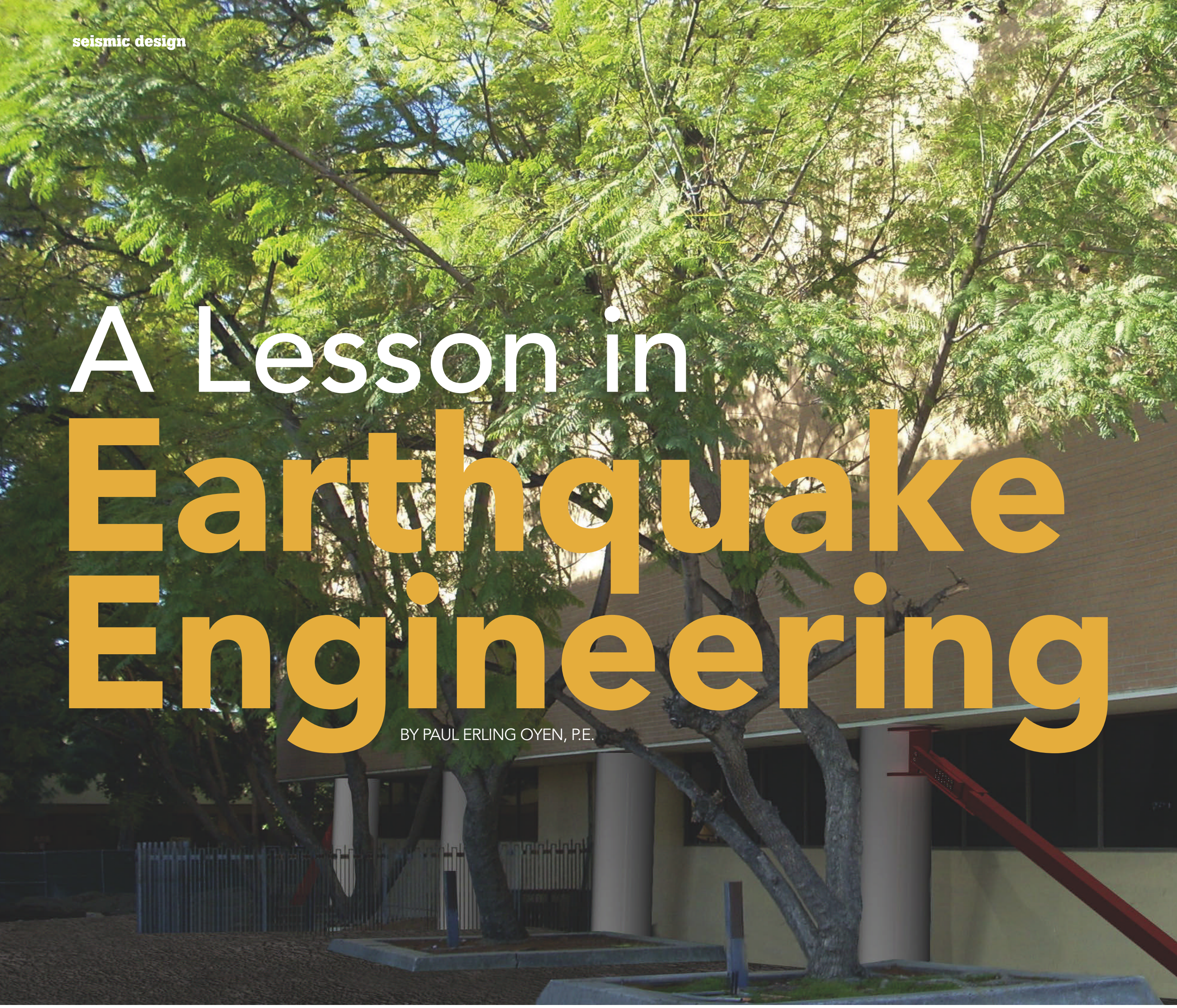
Turner-Smoot, a Joint Venture, Columbus

A Note on SPSWs

Examples of steel plate shear walls (SPSWs) as a seismic retrofit solution for low- and medium-rise structures can be found in the United States starting in the 1970s. Today, the system is being applied to a number of mid-rise and high-rise steel structures primarily on the West Coast. In this area of the country, many structures are undergoing seismic retrofitting to meet current codes, and SPSWs are extremely effective in resisting seismic and wind loads. However, though applicable, the system is not widely used in other parts of the country.

Typical steel plate shear wall systems consist of a steel plate wall, boundary columns, and floor beams. The steel plate wall and boundary columns jointly act similarly to a vertical plate girder. The steel plate wall itself acts as the web, and the horizontal floor beams act as transverse stiffeners in a plate girder. When loaded, the plate will experience large inelastic deformations, while the VBEs (vertical boundary elements) and HBEs (horizontal boundary elements) must remain elastic. This also needs to be the case under forces generated by fully yielded webs. Thus, the actual-versus-theoretical plate yield strength becomes extremely important in the design of the system. Recent studies have shown that the ratio of expected yield stress to specified minimum yield stress, R_y, for ASTM A36 plate material is 1.3 rather than 1.1, as specified in previous codes. These new findings significantly increase design loads on the system's VBEs, anchor bolts, and foundations.

An SPSW in the library's stack tower.



A Lesson in Earthquake Engineering

BY PAUL ERLING OYEN, P.E.

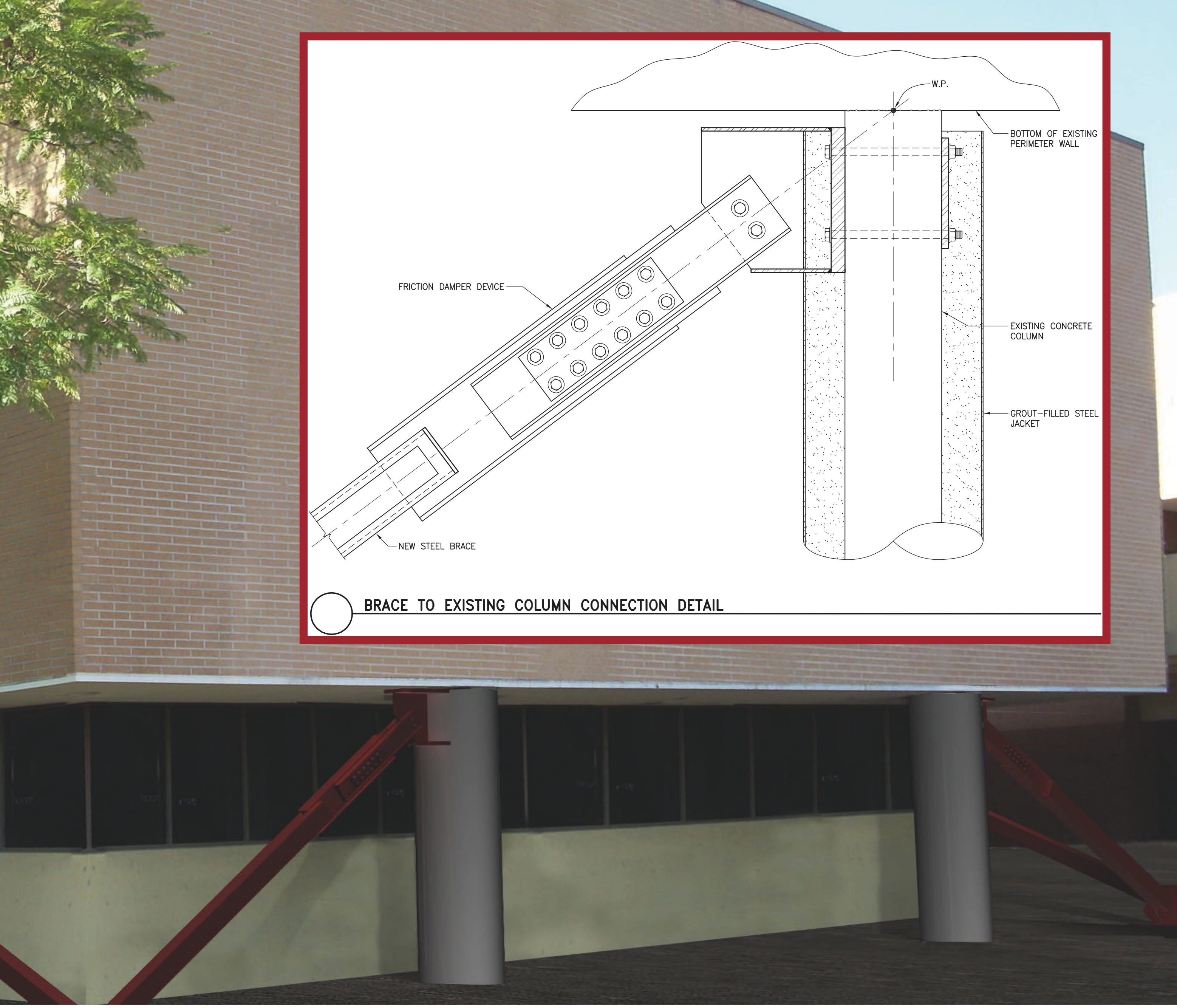
Steel bracing picks up the seismic slack for a concrete high school building in southern California.

BUILDING 1 OF THE GLENDALE HIGH SCHOOL campus (in Glendale, Calif.) is not unlike other high school buildings in the area. Built in the late 1960s, the two-story brick structure houses administrative offices and the school library. However, one element that will set it apart from others of its type is the exposed seismic bracing that will become an identifying characteristic of the building, once a seismic retrofit is completed this summer.

The roof is a cast-in-place slab over steel beams and tapered steel girders. The roof girders sit on second-story reinforced concrete and masonry bearing walls, and the second floor is a reinforced concrete pan-joist system supported by first-story reinforced concrete columns. There are no structural walls in the first story and no first-story columns at the corners, allowing the second-story walls to cantilever one full bay. The first floor is a reinforced concrete slab-on-grade, and the foundation of the structure is comprised of 18-in.-diameter, 30-ft-deep drilled piers in groups of two to four at each column. Grade beams tie the pile caps together.

Following the 1994 Northridge Earthquake, a review of the campus identified Building 1 as seismically deficient because of the soft story condition in the first story. Above the first story's nonductile concrete moment frames are full-perimeter concrete and masonry shear walls with few openings. The second-story system is much stiffer than the first story. In the event of a significant seismic event, displacement and damage would be localized to the first story and prevent energy from dissipating throughout the structure. Non-ductile detailing exacerbates the condition.

Typical of 1960s construction, the existing concrete columns have non-seismic reinforcement ties that are not spaced closely enough to provide sufficient concrete confinement. Additionally, some of the existing concrete frame beams are shear-critical under lateral loading. These details prevent the building from performing in a ductile manner and dissipating energy during strong seismic shaking. As such, the building would most likely not survive the shaking of a code-level earthquake.



Steel braces—HSS8×8×⅝ with two ¾-in. by 5-in. side plates—provide seismic reinforcement for Glendale High School. Images: Courtesy SGH

Eliminating Deficiency

To bring the building into the post-Northridge world, the Glendale Unified School District initially approached structural engineering firm Simpson Gumpertz and Heger with the task of eliminating the soft-story deficiency. The "obvious" solution was to add concrete shear walls to the first story. However, this option proved unacceptable, as the walls would prevent natural light from entering the building, and the overall result would not be aesthetically pleasing.

Another option we explored for eliminating the soft story was to use conventional steel braces to stiffen the first story. This solution required W14×311 braces for stiffness, adding several drilled piers to existing pier groups, and making difficult connections between the braces and the existing structure. This proved to be infeasible as well, as connections with the code-required overstrength forces could not be made to the existing pile caps.

Handling Displacement

We eventually settled on a displacement-based design approach using ASCE 41, *Seismic Rehabilitation of Existing Buildings*. This approach didn't eliminate the soft story but rather improved the structure's ability to safely handle the expected large displacements at the first story. The method determines the deformation demands at the first story from the design-level earthquake. The structural performance is predicted by comparing computed inelastic deformations to acceptable deformation limit states. The design objective for a seismic rehabilitation is to make the imposed seismic deformations stay within the limits. This can be done by decreasing the deformations (stiffening the structure) or increasing the limits (adding deformation ductility). Both were done to Building 1.

We reduced the displacement demands on the structure by stiffening the first story, though not enough to eliminate the code-defined soft story. This was accomplished by adding steel braces—HSS8×8×⅝ with two ¾-in. by 5-in. side plates—at each of the end bays (eight total). The weight of each brace frame was reduced from 8,000 lb to 1,500 lb. Accounting for the additional steel in the column jackets, the total steel weight went from 64,000 lb to 30,000 lb.

We increased the building's displacement capacity by selectively cutting existing interior column reinforcement bars, jacketing all first-story columns with grout-filled steel shells, and incorporating friction dampers in each new steel brace.

Though counterintuitive, cutting existing reinforcement bars in the concrete columns protects the concrete frame beams from catastrophic shear failure. The weakened columns act like a fuse. Under lateral loading, the columns will now hinge plastically prior to exceeding the shear capacity of the beams. The grout-filled steel shells provide confinement at these plastic hinge locations to considerably increase the rotational capacity of columns, thus increasing the displacement capacity of the structure.

Dual Functionality

In this design the friction dampers have two functions. They allow the braces to deform axially for the full design target displacement of the structure without buckling, and they limit the amount of force that can be imparted on the foundation and the connection to the existing structure. Friction dampers are essentially two plates of steel sandwiched together by pretensioned bolts in long slotted holes. The tension in the bolts and the faying surface between the plates is calibrated to allow the plates to slip under a specified load. The friction dampers in this rehabilitation are designed to slip at 200 kips, providing +/- 4.5 in. of displacement capacity. They are attached to the top of the existing concrete columns by through-bolted steel gusset plates. Each brace sits on new concrete pile caps over drilled piers at each corner. These pile caps are tied to the existing pile caps with new grade beams, and no additional foundation work was necessary.

Friction dampers provide a seismic segue between the new steel braces and the extisting columns.

Again, the rehabilitation design was performed to the ASCE 41 standard, which uses the concept of performance-based engineering. The performance objective for this building is the "Basic Safety Objective," which corresponds to the implied performance of a code-based design for an ordinary building. We performed nonlinear pushover analysis to evaluate the seismic demands using CSI's PERFORM-3D. We modeled existing concrete moment frames with steel shells as nonlinear frame elements and braces with friction dampers as elastic perfectly plastic axial springs with a yield force equal to the specified 200-kip slip load. Second-story shear walls were elastic shell elements, and the forces in the walls showed that they remain elastic under seismic loads. MSC

Paul Erling Oyen is Staff II – Structures with Simpson Gumpertz and Heger, Inc.

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Structural Engineer
Simpson Gumpertz and Heger, Los Angeles

Friction Damper Manufacturer
Pall Dynamics, Ltd.

Iconic Upgrade

The Church of Jesus Christ of Latter-day Saints addresses the nearby seismic fault with an upgrade to its iconic Tabernacle.

BY CRAIG WILKINSON, P.E., AND JEFF MILLER, S.E.

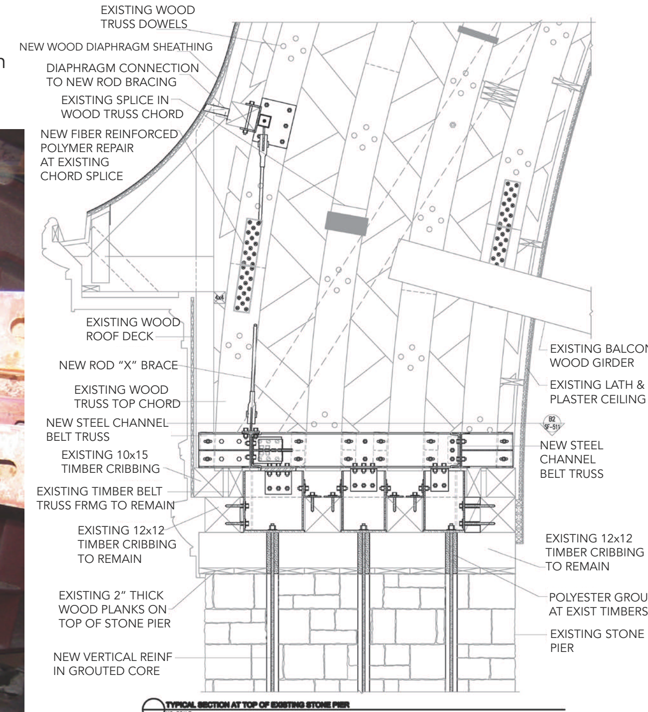
THE TABERNACLE IN SALT LAKE CITY has been a treasured piece of history for the members of The Church of Jesus Christ of Latter-day Saints since 1867. However, due to the Wasatch fault that runs along the foothills of the Salt Lake Valley, the Church wanted to protect its occupants and preserve the Tabernacle if and when a major earthquake occurs. Hence, nearly a century and a half after its completion, a seismic upgrade was performed on this home for the Mormon Tabernacle Choir.

The structure of the Tabernacle is composed of sandstone piers and wood trusses. There are 44 piers around the perimeter, each supporting a long-span timber arched truss. Each pier is 3 ft wide by 9 ft long, and the piers vary from 12 to 21 ft in height. The roof is framed with nine identical wooden arched trusses spanning between the stone piers over the main "barrel" section of the roof. At each end are 13 half-arch (radial) trusses that are supported by the last arched truss, the king truss, at the top and by stone piers at the bottom. The existing trusses are 9 ft deep and the full arches span approximately 150 ft. These trusses are composed of four chords of four 2½-in.

Steel segments for the truss had to be sized to fit through small hatches in the attic of the building.

opposite page and below: The new steel king truss complements an old wooden truss system, which had become overstressed and fractured over the years.

Images: Courtesy of Reaveley Engineers + Associates



above: A detail of the new belt truss system.

below: The exterior of the Tabernacle is composed of sandstone piers.

by 12-in. timbers. Each chord, consisting of two timbers on each side of a lattice web, is constructed of similar-sized timbers.

These trusses have had problems over the years with overstressed chord members fracturing and requiring repair. The king trusses were found to be severely overstressed due to the loads imposed on them by the radial trusses.

Steel in Sandstone

The major challenge was to seismically upgrade the structure while still preserving the historic integrity of the building, and several complex conditions had to be met with innovative structural engineering solutions. One of the biggest deficiencies of the structure in resisting seismic loads was that the timber members were not tied to each other or to the stone piers. A steel "belt truss" composed of more than 380 steel members with steel channel chords (MC12×35) and steel double-angle webs (2-L3½×3½×5⁄16) was designed. This oval-shaped truss rests upon the stone piers and extends around the entire perimeter of the roof structure. This belt truss, with a total inside "circumference" of about 620 ft, ties all of the existing timber trusses together and connects them to the sandstone piers.

This truss was designed to be constructed in segments small enough to be lifted in place through existing 30-in. by 30-in. attic hatches. The truss pieces were assembled via bolting, as cutting and welding in the highly flammable dry timber surroundings was not feasible. Precise measurements were required to ensure that all of the bolted connections would fit.

New steel king trusses were designed and installed adjacent to the original timber king trusses due to a high level of overstress in these trusses. The new trusses were used to lift the existing trusses and re-support their load. Each of the new steel trusses is approximately 7 ft, 1 in. deep and spans 140 ft. Similar to the belt truss, the design and fabrication had to be very precise, as the finished trusses had to be assembled together in the attic, again without welding or cutting any members in place. Small sections of the roof were temporarily opened to insert the pieces of steel truss without disturbing the existing trusses. This process was undertaken in several phases to ensure no harm was done to the existing structure.

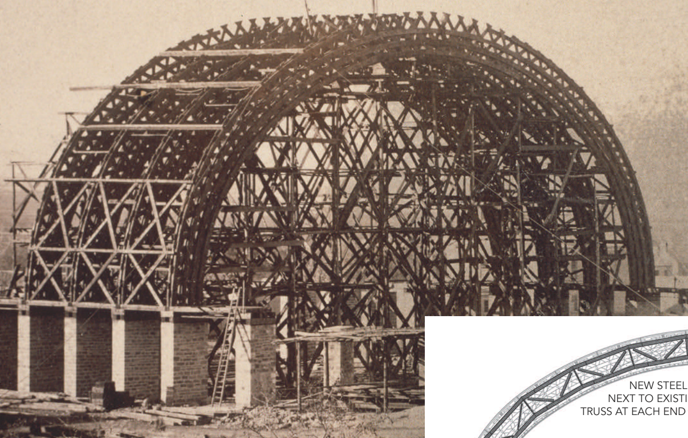
The design of the steel king truss presented two major engineering challenges. The first was controlling the amount of horizontal thrust imposed by the arch on the supporting structure. With the shape of the arch being constrained to fit within the envelope of the attic, the truss could not be reconfigured to reduce the thrust. Through extensive modeling and analysis, it was found that allowing the base of the arch to spread outward was an effective means of allowing the new arch truss to "relieve some stress" and reduce the thrust. The design was optimized to allow each support to move 3 in. outward before being restrained and locked into place. The top and bottom chords of the truss were designed to intersect at a single point and placed on Teflon slide bearings that were restrained from moving more than the allowed 3 in.

The second challenge was detailing the top of the truss where the existing timber trusses were to be lifted a small amount. The lifting of the existing trusses was essential to guaranteeing their load was captured by the new steel trusses. A detail was developed that allowed the new truss to deflect downward while the existing truss was lifted slightly upward. Hydraulic jacks were used to lift the existing trusses and load the new truss. The sliding mechanism was then "locked" to permanently support the load, and the jacks were removed.

When it came to the piers, each was strengthened by coring vertical holes and reinforcing them with grout and high-strength steel threaded rods. The sandstone foundation for each sandstone pier was also strengthened by encasing the piers with reinforced concrete and adding micropile foundations.

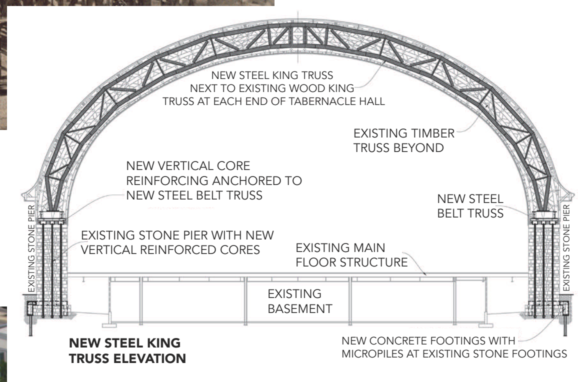
Numerous constructability and sequencing reviews with the team were critical to designing and specifying the improvements to the Tabernacle in a way that would allow the construction team to achieve maximum efficiency, while at the same time staying within the strict project constraints to preserve the historic fabric and protect the building. Today, the structure retains all of its majesty—with the added bonus of being seismically safe. MSC

Craig Wilkinson is a principal and Jeff Miller is a senior principal, both with Reaveley Engineers + Associates, Salt Lake City.

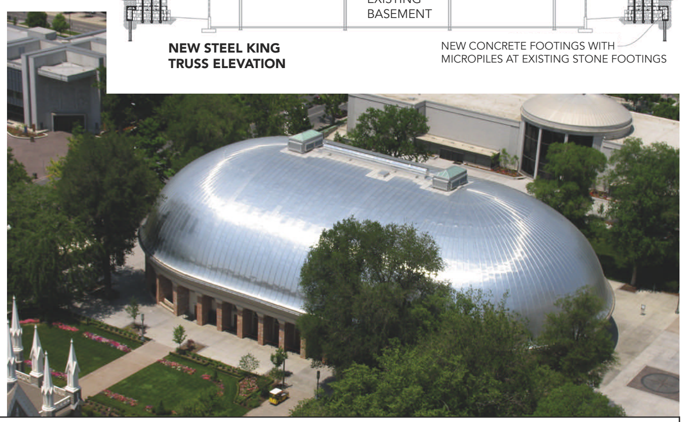


above: The framing of the Tabernacle as it appeared in 1867.

below: Present-day framing of the new steel king truss.



right: The Tabernacle has been an icon of The Church of Jesus Christ of Latter-day Saints for a century and a half.



Images: Courtesy of Reaveley Engineers + Associates (right and above right); Courtesy of The Church of Jesus Christ of Latter-day Saints (above)

Owner
The Church of Jesus Christ of Latter-day Saints, Salt Lake City

Architect
FFKR Architects, Salt Lake City

Structural Engineer
Reaveley Engineers + Associates, Salt Lake City

Steel Fabricator
S&S Steel Fabrication, Inc., Hurricane, Utah (AISC Member)

Steel Detailer
Computer Detailing, Inc., Salt Lake City (AISC Member/NISD Member)

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BRBFs provide superior seismic response, but SCBFs are still a viable bracing option in many seismic applications.

Ample Seismic Protection

BY CARLOS DE OLIVEIRA AND JEFFREY A. PACKER, PH.D., P.ENG.

SOME EXCELLENT RESEARCH on special concentrically braced frames (SCBFs) is making its way from university laboratories all over North America to the drawing tables of consulting engineering firms all over the world.

The research, some of which has yet to be published, is reminding us that, yes, buckling-restrained braced frames (BRBFs) provide improved seismic response over SCBFs—but with appropriate brace member selection and detailing, SCBF structures without buckling-restrained braces also can provide ample seismic protection. And when you factor in simplicity in design and construction, it's no wonder that SCBFs still have their place as the preferred lateral force resisting system in medium- to low-rise steel structures. This is especially true in design-build projects, where involving additional consultants and meeting stringent testing requirements, as required for BRBF structures, can add too much additional time to a project.

Cyclical in Nature

Unlike with BRBFs, the bracing elements in SCBFs are meant to cyclically yield in tension and buckle in compression in an earthquake. In fact, the cyclic brace yielding and buckling is how the system absorbs energy imparted by strong ground motion. As such, the cross-sectional shape, cross-sectional slenderness, and overall slenderness of the brace members in an SCBF determine the building's overall response in an earthquake. So it's no wonder that the effects of these parameters on the response of SCBF are being studied by so many researchers and are the subject of such hot debate within code committees.

In 2007, Uriz, Sabelli, and Mahin submitted a report on the design implications of the preliminary results of ongoing research on SCBF systems to AISC. Since then, as can be seen in a recently published draft of the AISC 2010 *Seismic Provisions*, the AISC Technical Committee 9 (Seismic) has adopted the Uriz et al recommendation to tighten the permitted cross-section slenderness of round hollow structural sections (HSS) to $D/t \leq 0.038\ E/F_y$ from the previous limit of $D/t \leq 0.044E/F_y$.

In 2006, Packer suggested the use of round HSS over rectangular HSS for energy-dissipating braces. New research by Fell, Kanvinde, Deierlein, and Myers published in the January 2009 ASCE *Journal of Structural Engineering* supported this and suggests that wide-flange sections and round HSS or pipe braces provide a more desirable SCBF response than rectangular HSS braces. Fell et al., point out that in these superior sections, local buckling occurs more gradually and thereby delays fracture initiation at the central plastic hinging point of the brace.

From a practical standpoint, the Fell et al., results can be applied by engineers today. It is commonly accepted that HSS are efficient for carrying compressive loading. Since we must size SCBF bracing elements to carry compressive forces, it makes sense to specify round HSS (produced to ASTM A500 or CAN/CSA G40.20/21) or pipe (produced to ASTM A53) bracing elements whenever possible. Once the compressive forces become too large to be carried by round HSS or pipe elements (i.e., the axial compressive capacity of the available sections that meet the stringent cross-sectional and overall member slenderness requirements for SCBF is not sufficient), then we should move to wide-flange braces.

Connections for Round Hollow Braces

Regardless of the cross-sectional shape of the steel brace section used in a SCBF, brace connections must be detailed such that they can resist tensile loads greater than or equal to the expected yield strength of the brace (given by $R_y\ F_y\ A_g$). A slotted HSS-to-gusset connection is the most common detail used for connecting HSS brace members to the beam-column intersection. This type

Welding a Cast ConneX High-Strength Connector to a round HSS brace element for use in an SCBF.

Photos (this page and next): Carlos de Oliveira

of connection induces shear-lag in the hollow section, which can lead to connection facture at loads that are lower than the expected yield strength of the brace. Thus, the AISC *Seismic Provisions* require the addition of net-section reinforcement in slotted hollow section bracing connections.

As discussed above, round HSS or pipe elements provide better energy-absorbing bracing than do rectangular HSS, but the reinforcement of round sections requires the use of curved plate, channels, angles, or segments of other round sections, which can make detailing and fabricating the reinforced connection more onerous. Further complicating the issue, the slots that are cut or burned into the HSS itself must have smooth edges, as notches in the slots can become sites for crack initiation and propagation in the connection during an earthquake.

Commonly, field welding of the demand-critical fillet welds between the slotted HSS and gusset is specified, which can be costly and requires substantial quality control and field inspection. If field-bolting is desired, the connections must be spliced as the load path must remain concentric, thus requiring a significant number of bolts, all of which must be pretensioned. In many cases, the number of bolts required is prohibitive.

Cast ConneX High-Strength Connectors offer a practical solution for SCBF brace connections for round HSS or pipe. The connectors, which are shop-welded to the bracing, allow for bolted installation of the brace members. They accommodate a double-shear bolted connection, so the number of bolts required can be drastically reduced. Alternatively, if site fit-up becomes an issue, or if the connections are to be exposed and the architect would like to avoid bolted connections, the connectors can be field-welded to the gussets.

These connectors were developed by Packer, Christopoulos, and de Oliveira at the University of Toronto (de Oliveira et al., 2008) and braces equipped with these connectors have undergone testing at both the University of Toronto and Montreal's Ecole Polytechnique. (For more information, visit **www.castconnex.com**.)

Ongoing Research

Hold on to your hats, because a lot more SCBF research is currently underway. Watch for more publications by Roeder (Washington), Kanvinde (Univeristy of California–Davis), Deierlein (Stanford), Tremblay (Ecole

Full-scale cyclic testing of a round HSS brace assembly equipped with High-Strength Connectors at Ecole Polytechnique in Montreal.

A SCBF with round HSS bracing equipped with High-Strength Connectors.

Polytechnique, Montreal), and Packer (Toronto), just to name a few. These studies will no doubt improve our understanding of SCBFs and should shed more light on how practicing engineers should be designing seismic-resistant braced structures for optimal performance. Also, the AISC HSS Committee—which includes HSS producers, steel fabricators, and educators/researchers that actively participate in the design and use of HSS—has been working on developing a new higher performance manufacturing specification for HSS. This is being submitted to ASTM this year and is intended to be another alternate production standard, *in addition to* ASTM A500. This will feature tighter control of geometric tolerances and improved mechanical properties (including a notch toughness requirement and a restricted yield stress range), thus making it ideal for seismic design. MSC

Carlos de Oliveira is CEO of Cast ConneX Corporation, and Jeffrey A. Packer is a professor at the University of Toronto.

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In the Moment

BY VICTOR SHNEUR, P.E.

Here are 60 tips for simplifying fully restrained moment connections for W-shapes.

MANY BUILDINGS HAVE MOMENT CONNECTIONS FOR lateral frames and/or cantilevers. Even though they don't encompass the majority of the connections, it is important to give moment connections special attention, since they require more work and may be a safety consideration during erection.

Here are a few (well, 60) suggestions for making moment connections easier to design, detail, fabricate, and erect, along with a few recommendations for avoiding problems:

1. Moment connections for cantilevers require special attention from the erector. They always must be completed, including moment connection for backing beams, before the cantilever is released. Otherwise, adequate temporary support should be provided. Also, per OSHA requirements, a competent person should supervise cantilever erection.

2. If moment connections are required for the lateral load resisting system, select an R of 3 or less whenever possible. When $R > 3$ the AISC *Seismic Provisions* must be applied, which has a significant associated cost implication.

3. When heavy rolled W-shapes are required at moment connections with complete-joint-penetration (CJP) groove welds, don't forget about special requirements for the material covered in the AISC *Specification*, Section A3.1c.

Victor Shneur is chief engineer with LeJeune Steel Company in Minneapolis.

4. When moment connections are not designed by the EOR, provide all end reactions, including vertical end reactions and moment envelops. The fabricator can then select the most efficient connections and check columns for reinforcement.

5. For non-domestic sections, consider using A913 steel to substantially reduce preheat requirements at welds (see Table 3.2 in the AWS D1.1:2008) and possible column reinforcement at moment connections.

6. As the engineer of record, request removing backing bars *only* when required by the governing code or architectural reasons. (This is an expensive procedure.)

7. Do not fill weld access holes with weld material for cosmetic or corrosion-protection reasons. In addition to the cost, it creates undesirable triaxial stresses. Using mastic materials is preferable to welding.

8. Avoid weak-axis moment connections at W-columns.

9. For moment frames, consider using partially restrained or flexible moment connections in lieu of fully restrained connections whenever possible.

10. Carefully examine cantilever framing for reducing the number of members with moment connections. This is the best way to save money on moment connections. Potential increase in material weight can be well justified by savings in labor and safer erection.

11. When a direct-welded flange moment connection is made to a column web, extend connection plates at least ¾ in. beyond the column flanges to:
    - ➜ avoid intersecting welds
    - ➜ provide for strain elongation of the plates
    - ➜ provide adequate room for runout bars

12. When possible, consider using a deeper W-shape to reduce flange forces and possibly eliminating stiffeners at columns. The increase in material weight is typically offset by eliminating stiffeners and using a less expensive/lighter moment connection (e.g., an extended end-plate connection in lieu of a directly welded connection if end moment allows).

13. When moment connections are made to a column or beam web, use beams with the same depth on both sides of the web where possible.

14. Don't specify fully restrained moment connections to resist moderate beam axial forces. Double-angle connections with thicker angles for perpendicular framing and shear end plate connections with thicker plates for skewed framing often can be designed for combined shear and tension forces.

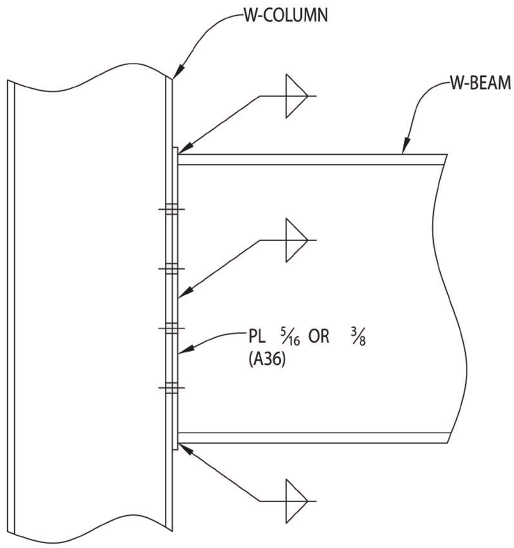


Figure 1. End-Plate Connection for Torsion

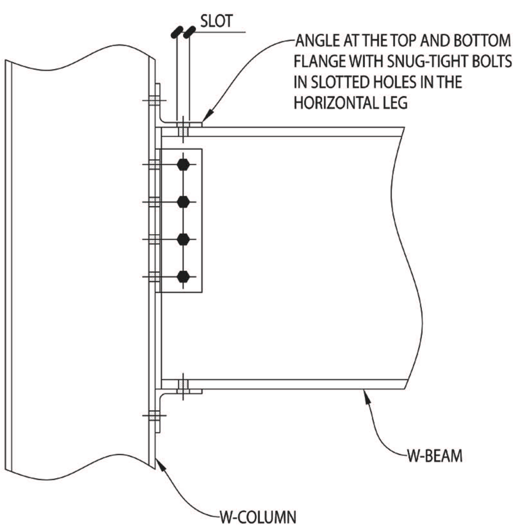


Figure 2. Flange-Angle Connection for Torsion

15. Don't use fully restrained moment connections to resist torsion. Typically, a 5⁄16-in. or 3⁄8-in. end plate shop-welded to both flanges or bolted flange angles will provide adequate strength. Note that connection flexibility can be provided by keeping bolts at the end plate between the flanges, or using snug-tight bolts in the slotted holes in horizontal legs of flange angles. (**Figures 1** and **2** illustrate these connection concepts.)

16. Check section dimensions (depth, flange width, flange thickness) at moment connections. Choosing sections that fit correctly may simplify the connections (e.g., CJP welding detail at the bottom flange when flanges match each other).

17. Remember that eccentricity can be neglected in the web shear connections. As explained on page 12-3 in the 13th edition AISC *Manual*, it is permissible "since, by definition, the angle between the beam and column in a fully restrained moment connection remains unchanged under loading."

18. Bearing bolts in standard or horizontal short-slotted holes (perpendicular to the line of force) are permitted in the web shear connections. This can reduce number of bolts and avoid special requirements for faying surfaces.

19. If beams with moment connections frame into both column flange(s) and web, try to use the same depth for all beams. This eliminates interference where stiffeners are required.

20. Skewed moment connections at columns, especially for beams framing into a column web, can be difficult to make. Modifying the framing, rotating the column, or slightly moving the beam end can greatly simplify these connections.

21. Pay attention to sloping beam-to-beam moment connections; they require special load analysis due to the vertical component of the flange force. Also, the connection layout is typically more complex.

22. Avoid cambering beams with moment connections, because moment connections provide end restraint and reduce deflection. As L.A. Kloiber, P.E. explained in the article "Cambering of Steel Beams" (MSC, 1989), "Moment connections such as end-plate connections, top-and-bottom-plate connections, and direct-welded connections will not fit up properly unless the connection face is fabricated vertical. This requires special layout and cutting after cambering and is an added expense."

23. Shop-weld short cantilevers to the column as shown in **Figure 3**. This will make the erection much safer.

24. Cantilever framing to a column doesn't need a backing beam with a moment connection on the other side of the column when the column has adequate strength to resist the cantilever moment.

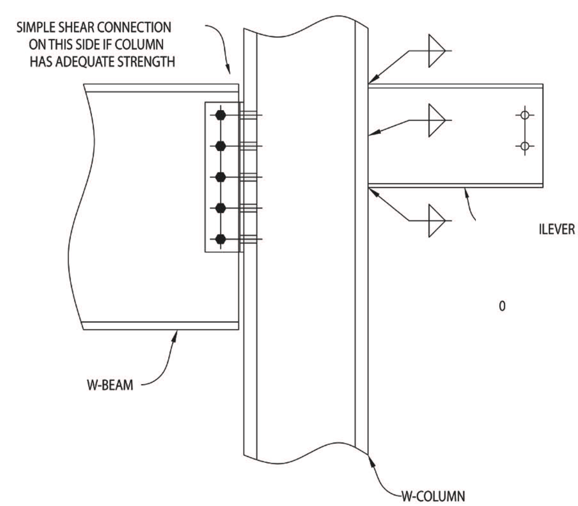


Figure 3. Shop-Welded Short Cantilever

25. If the cantilever moment needs to be balanced, review the effect of the backing beam moment connection. A partially restrained moment connection may be used to reduce the end moment delivered to the column.

26. When the cantilever is required at the roof, making the beam continuous over the column will eliminate the moment connections for the cantilever and backing beam. This will make fabrication and erection easier and safer.

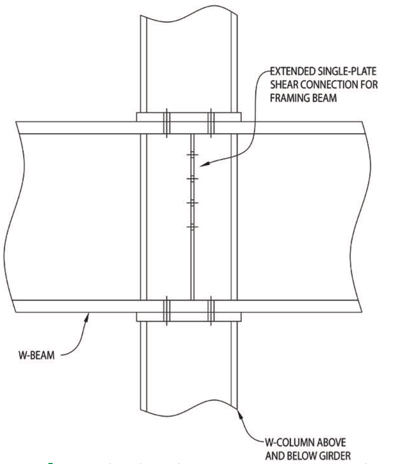


Figure 4. Stacked Column at Large Cantilever

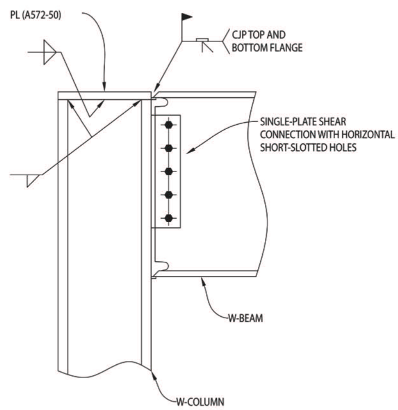


Figure 5. Directly Welded Moment Connection at Top of Column

27. For a large floor cantilever/beam, consider stacking the columns as illustrated in **Figure 4**. This may be easier than reinforcing the column supporting the cantilever and having two large moment connections in the field.

28. When a directly welded moment connection is made at the top of the column in lieu of welding a beam top flange to the column flange and providing two stiffeners between column flanges, make one cap plate and weld the top flange directly to this plate (see **Figure 5**).

29. When possible, favor extended end-plate moment connections over directly welded moment connections. AISC Design Guide 4, *Extended End-Plate Moment Connections: Seismic and Wind Applications* provides design procedures and recommendations. Extended end-plate moment connections make the erection much simpler and safer, eliminating CJP welds at flanges in the field.

30. At extended end-plate moment connections for non-seismic applications, it is acceptable to weld flanges with fillet welds on both sides in lieu of all-around welding, when adequate strength is provided.

31. At cantilever-to-beam connections, when the bottom flange is always in compression, use an end-plate connection extended below the bottom flange as illustrated in **Figure 6**. In this case, top-flange tensile force will be resisted by a CJP weld or flange plate, and bottom-flange compressive force will be resisted by bearing. Any field connection (CJP weld or flange plate) is eliminated at the bottom flange. The same concept can be applied to:

➜ Cantilever and backing beam-to-column

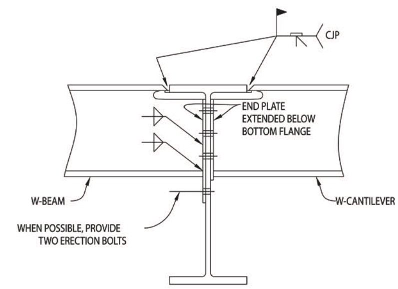


Figure 6. Cantilever Moment Connection at W-Beam when End Moment is not Reversible (Bottom Flange is Always in Compression)

moment connections when the bottom flange is always in compression
- ➜ Field splices for beams and plate girders when the top flange is always in compression

32. Consider using heavier column sections to eliminate the reinforcement (stiffeners and doublers) at moment connections. Chapter 3 in AISC Design Guide 13, *Stiffening of Wide-Flange Columns at Moment Connections: Wind and Seismic Applications* provides suggestions and cost comparisons.

33. Layout welds to reduce restraint, especially for large welds. This lowers the possibility of lamellar tearing.

34. Favor fillet welds over groove welds.

35. Allow full-strength connections in lieu of CJP groove welds in statically loaded structures (e.g., for welding stiffener plates at columns). Fillet welds up to ¾ in. are more economical.

36. Avoid field-welded moment connections for galvanized members, especially when end moments are not large. Galvanizing requires special ventilation in closed areas and usually needs to be removed and restored.

37. Normally, a directly welded moment connection is preferable at rectangular hollow structural section columns. If possible, increase HSS column wall thickness and/or use a deeper W-shape to reduce flange forces to eliminate expensive through plates at W-flanges welded directly to the HSS wall.

38. If, however, horizontal through plates are required due to large moments and moment connections are made to different sides, use same-depth beams to eliminate multiple through plates at bottom flanges.

39. Consider flange-plated moment connections to round HSS/pipe columns or to the corner of rectangular HSS columns. Directly welded moment connections may create erection clearance problems when top and bottom flanges are prepared to match the supporting column shape (unless the connection at the other end allows bringing the beam in).

40. When rolled beams and plate girders need to be field-spliced, use end-plate connections described in AISC Design Guide 16, *Flush and Extended Multiple-Row Moment End-Plate connections* when possible.

41. Moment connections to embedded plates in concrete require special details because of the different tolerances for steel and concrete. When designing these connections:
- ➜ Make embedded plates larger than required for connections to allow for concrete tolerances.
- ➜ Size embedded plate thickness conservatively; it may be moved from the design position, and flange tensile force will not be applied at the theoretical location.
- ➜ Headed studs are preferable to transfer beam flange tensile force. When large moments need to be resisted and long anchors/rebars are required, consider using anchors that are field-attached to the plates or field-screwing anchors into the couplers shop-welded to the plates. This will make fabrication and installation easier.
- ➜ All connection material needs to be field welded to the embedded plates because of interference with formwork.
- ➜ Flange-plated connections field-welded to both the beam and embedded plate are preferred because of much tighter tolerances for steel than for concrete members.

42. Choose the geometry of preparation for CJP groove welds to minimize

weld metal volume. This reduces labor, shrinkage, and the possibility of discontinuities.

43. Design moment connections to eliminate overhead welds in the field. For example, when moment flange plates are field-welded, make the top flange plate narrower than the flange and the bottom flange plate wider than the flange (see recommended minimum shelf dimensions as shown in Figure 8-11 in the AISC *Manual*).

44. When using a field-bolted top flange plate, make a note to provide deck bearing at the flange connection. A ¼-in. shim between plate and flange can be extended providing support in lieu of a standard deck angle. **Figure 7** shows an example with a ¼-in. shim.

45. Remember that a CJP groove weld in a directly welded flange connection can be expected to shrink from 1⁄16 in. to 3⁄16 in. in the length dimension of the beam when the weld cools and contracts. It is especially important when a multi-bay moment frame with CJP groove welds is used. This should be coordinated with the fabricator and erector to establish the appropriate connection detail and erection procedure.

46. Mill tolerances for beams and columns may cause significant misalignments of holes in flange-plated connections when bolts groups are large. Consider shipping the flange plates loose for field welding. This also eliminates additional shimming at these plates.

47. Always design and detail connections for the tolerances. At every moment connection, the web and both flanges of the framing beam are connected to the supporting member. Disregarding tolerances may make connections unworkable and lead to costly modification. Refer to ASTM A6/A6M, AISC *Code of Standard Practice for Steel Buildings and Bridges*, and AWS D1.1 for the allowable mill, fabrication, and erection tolerances. Depending on actual

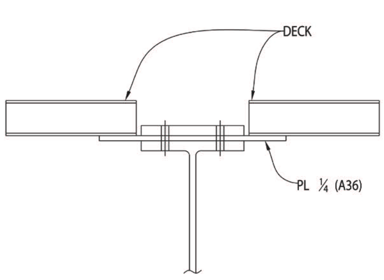


Figure 7. Deck Bearing at Bolted Flange-Plate Connection

connections, there are a number of different ways to provide for tolerances. For example, for directly welded flange-to-plate connections at column webs, specify connection plates thicker than the flanges; use slip-critical bolts in oversized holes for flange-plated connections, etc.

48. And never forget constructability and clearances for welds and bolts. For example, when a directly welded moment connection is made to a column web, locate the bolt group for the web connection outside of the column flanges. This simplifies erection and bolt pretensioning and reaming, if required.

49. As the engineer of record, ask a local fabricator or erector for his/her advice in cases of special situations. This can save time and money down the road, especially for repetitive connections.

50. Remember that inspection immediately drives up the cost and needs to be specified carefully and only as needed. For example, welds that are subject to low stresses or are in compression don't need the same inspection as welds subject to high tensile stresses.

51. As the engineer of record, unless they can be justified for unique conditions, avoid specifying more stringent requirements than established by standard practice and included in the current specifications, standards, codes, and provisions. All procedures have been developed to meet requirements per these documents, and more stringent requirements will lead to establishing new procedures and cost increases (especially when the number of bidders will be reduced).

52. When a new moment connection is made to an existing frame, carefully examine existing conditions including actual plan dimensions, elevations, member sizes, steel grades, steel weldability, etc. Keep in mind that: members could be substituted; moment connections are always very sensitive to the tolerances; and typically it is difficult to reinforce existing members.

53. When sequencing steel frame erection, consider moment frames for stability. This may allow savings for temporary bracing.

54. Use correct sequences when making fully restrained moment connections. For example,
    - ➜ At directly welded moment connections at columns, pretension bolts at web the connections *after* the welds at the flanges are made and allowed to shrink. Otherwise, the weld shrinkage would cause significant amount of preload in the bolts and welds. Provide horizontal short-slotted holes in the web connections.
    - ➜ At moment connections with CJP groove welds at the web and flanges (e.g., beam splice), weld the web first to reduce additional stress due to restraint.

55. The perfect design will not eliminate all mistakes. Good connections substantially reduce the number of shop and field problems, but remember: people make mistakes, actual tolerances may be larger than expected, and problems may arise. As the engineer of record:
    - ➜ Request an as-built report. It clearly shows the problem and eliminates misunderstanding or misinterpretation.
    - ➜ Discuss possible solutions with a fabricator or erector; chances are they already have a suggestion.
    - ➜ Consider the cost; labor is expensive and material is cheap.
    - ➜ Proceed with fast decisions and approvals to continue the work.
    - ➜ Remember that not every field problem requires correction.

We're down to the last five. Here are some suggestions for moment frame connections when seismic provisions must apply:

1. Use prequalified connections included in AISC 358 *Prequalified Connections for Special and Intermediate Steel Moment Frames for Seismic Applications*, included in the AISC *Seismic Design Manual* (Section 6.2).

2. When heavy rolled W-shapes are required at moment connections with CJP groove welds, don't forget about special requirements for the material covered in the AISC *Seismic Provisions*, Section 6.3.

3. Detail weld access holes at CJP groove welds to comply with Tables 1-1 and 1-2 in the AISC *Seismic Design Manual*.

4. Provide items required per the AISC *Seismic Provisions* and Section 5 in particular.

5. As for non-seismic applications, favor extended end-plate moment connections over directly welded moment connections. Refer to AISC 358 *Prequalified Connections for Special and Intermediate Steel Moment Frames for Seismic Applications* for design requirements for these connections.

Moment connections vary greatly, loads can be large, and framing conditions can be complex. However, as with all other connections, the best effect is achieved when design, fabrication, and erection expertise are combined together. MSC

True Collaboration

BY DICK DECKER

The next generation in construction project delivery promotes cutting-edge technology and a truly joint effort early in the process.

THE NEXT STEP IN THE EVOLUTION of project delivery methods is upon us and goes by the name of Integrated Project Delivery (IPD). Collaboration between designers and constructors from a project's inception is the cornerstone of this method, which is relatively new yet has been increasing in popularity. What's the draw? Cost savings—big-time cost savings. The larger the project, the more savings an IPD approach can bring to the table.

So how did we get to this point? Let's step back in time a bit and take a brief look at IPD's precursors.

Design-Bid-Build

Design-Bid-Build (DBB) has been around for hundreds—some would argue thousands—of years and is the traditional accepted project delivery method. Owners liked the concept of competitive bidding and fixed price, while contractors liked the fair and equitable concept that everybody bids on the same job specifications. Architects and engineers liked it because they were the technical managers and set the project specifications for everyone to follow. Everybody was happy, right? Well, not exactly.

Owners started to realize that project costs were higher than they needed to be with the DBB method. Contractors had to put in extra contingency money for those unforeseen conditions like design changes, late project deliveries, and a few legal costs here and there; when all the project contingencies were added up it could result in 20% to 25% of total project costs! Then there were the change orders. (Has anyone ever done a "fun" change order? Not likely.) Also, some subcontractors resisted taking responsibility for coordinating work with other subs. Designers started to get uncomfortable being held responsible to produce the perfect design.

Design-Build

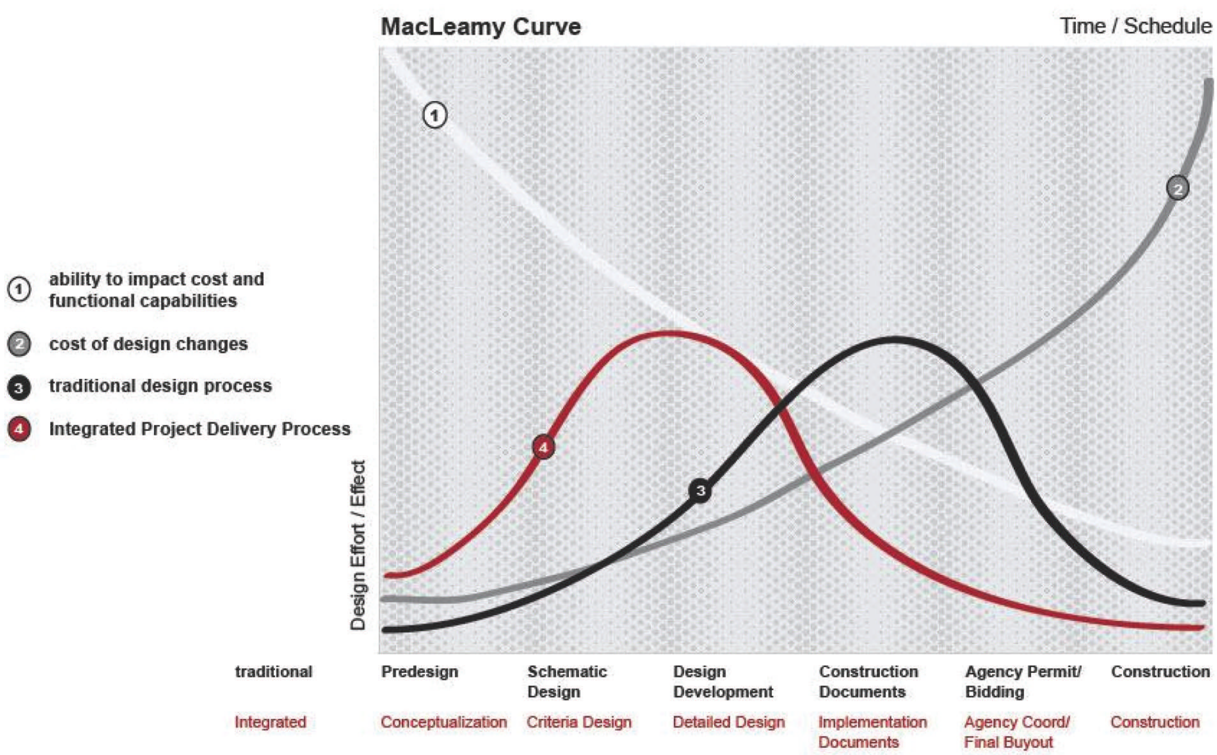
These deficiencies paved the way for the next step in project delivery evolution: Design-Build. DB allowed the general contractor to manage the complete project, usually including the designers. With DB, we had an experienced, knowledgeable entity in charge. There is no question that this increased efficiency over DBB. The successes can be measured in increased on-time deliveries and reduced costs. Clearly, the strength of DB was in the insertion of the experienced, knowledgeable project leader that was able to make informed project decisions in a timely manner. A number of GCs have been very successful with DB, as their ability to build and work with a team of people is critical to successful DB projects.

A number of GCs have taken the DB method a step further and used teams for collaboration in the initial design stage, thus creating a hybrid version of DB-IPD. However, some GCs, schooled in DBB traditional method, were staffed with managers and superintendents of the Type A, or Alpha, personality type. These "A" leaders encountered issues with developing functional, collaborative teams and as we all know, a non-functional team can fail to deliver projects on time and incur higher costs.

While DB was clearly an improvement over DBB, it did not eliminate all the problems; it just reduced them. It became very obvious that team building was probably the major determinant to a successful DB project. Getting teams to work together and collaborate—really collaborate—was the jumping off point from DB to the beginnings of IPD.

Integrated Project Delivery

IPD is the adoption of collaborative methods, starting in the design stage, to improve some of the remaining team-building



Introduced in the Construction Users Roundtable's "Collaboration, Integrated Information, and the Project Lifecycle in Building Design and Construction and Operation" (WP-1202, August, 2004)", the "MacLeamy Curve" illustrates the advantages of Integrated Project Delivery.

issues with DB. Younger managers and engineers coming out of technical schools were exposed to collaborative methods and brought those concepts with them into the industry. As the successes with collaborative team-building methods began to build, the AEC industry wanted to learn more about it. The management motivation to step into IPD was there, but project management changes cannot occur without an economic incentive.

Early this century, along came Patrick MacLeamy, CEO of architecture firm Hellmuth, Obata + Kassabaum (now HOK) and his now famous curve. The MacLeamy curve (above) indicates the reason why IPD adoption has an economic incentive. The graph clearly shows the time period of a project where the greatest cost reductions can occur: during the design phase of the project. If we make the design phase of the project more efficient by having the designers and constructors collaborate, then we get a more economically efficient project overall. The creative abilities of a team of intelligent people all focused on the benefit of the project as a whole, as opposed to their own silo, is an extremely powerful cost-reduction tool!

The white line on the graph, 1, indicates that the ability to impact cost and functional capabilities of the project is greatest in the beginning design stage and lowest in the construction stage. The gray line, 2, shows that the cost of design changes is low in the beginning design stage but very high in the construction stage. The black line, 3, indicates where the greatest amount of effort is expended in a traditional project, and the red line, 4, shows the shift in maximum effort with IPD: earlier in the project where costs are more easily reduced. The obvious conclusion is that we need to move the maximum effort into the design stage where the ability to reduce project costs are higher and the cost to make changes is much lower. This is the economic message of IPD.

True Collaboration

IPD is a collaborative project delivery method using relational contract principles to harness all of the strengths and capabilities of the owner, designers, and constructors and focus them on one goal: the efficient delivery of the project as a whole. Successfully implemented, IPD can take separate, individual AEC companies (silos) and turn them into efficient functional teams with creative problem-solving abilities that far exceed the successes of DBB and DB. The participating team members (subcontractors) are usually selected by a team consisting of the owner, managing contractor, lead designers, and other IPD team members. In the more successful IPD projects, team members are not selected by bid price, but rather by their industry experience and their ability to function in a collaborative team environment. The selected team members start meeting from the inception of the project as the design begins.

The birth of the IPD process has been largely credited to Westbrook Construction, a 57-year-old mechanical construction company in Orlando, Fla. Westbrook was a DBB contractor that moved into DB but still kept looking for a better way to use the creative abilities of the whole AEC team, not just their own employees. Another organization, IPD, Inc., also of Orlando, began developing relational contract forms as a non-profit corporation that existed only during the project and then distributed all its profits to the IPD team members at the end of the project. The evolution of IPD continues, unabated, worldwide.

Big IPD, Little IPD

IPD can be as elaborate or as simple as you desire. "Little IPD" consists of a team of designers and constructors meeting every so often to review 2D drawings. Collaboration occurs and designers and constructors give each other valuable feedback. Design improvements are made based on collaboration. In Little IPD, all the team members might not necessarily meet at one time. There are sub-groups that meet at different times to discuss their own particular areas of expertise, mechanical, electrical, and plumbing (MEP) being a primary example. However, it is difficult to harness all the collaborative benefits of an MEP group without a good clash-detection method. This is where we move into "Big IPD."

Big IPD brings in many more tools. The use of 3D modeling and building information modeling (BIM) is a foundation of Big IPD. BIM functions extremely well in a team environment because it fosters collaboration, the foundation of IPD. It allows remote team members to collaborate with the group from wherever they are located. Further, BIM's clash-detection capability can help a project save huge amounts of time and money when it comes to construction, particularly in the reduction of clashes between MEP and structural systems.

Another cutting-edge project tool that is integral to IPD is Lean Construction (and its Target Value Design concept), thanks to its new way of looking at eliminating waste and costs. I was recently involved in an IPD project that saved $60 million dollars in design and projected construction costs in one year using Target Value Design methodology. (See the November 2008 issue of MSC for an article on this project, "Lean Construction in California Health Care," and one on BIM, "Technical Solutions are Just the Half of It," both available at **www.modernsteel.com**.)

Can you build an IPD project without all of these tools? Sure you can, as is illustrated by Little IPD, which was implemented before the above tools came along (and still is). There is money to be saved using both Little IPD and Big IPD. However, design and construction team managers that have experienced both types will tell you that the more tools you learn the more fun the project becomes. And the desire to learn is an absolute must for anyone considering doing IPD.

IPD Agreements

Now that you have a better idea of what IPD and its advantages are, you're probably wondering how/where it exists on paper. One such document is ConsensusDOCS300, which was introduced in September, 2007 as an IPD relational form of agreement. ConsensusDOCS is an alliance of more than 20 AEC firms and organizations, including the Association of General Contractors (AGC), and is the first IPD delivery contract developed using IPD principles of collaboration and consensus. The basis of this agreement is collaboration and risk-sharing between parties. Since it was developed by industry participants, it takes a more "even-handed" approach and attempts to not to shift risk to any particular participant over another. It establishes a core group of owner, architect, and general contractor to be the overall management leadership of the IPD project. Last year, ConsensusDOCS released the ConsensusDOCS301 BIM addendum, which defines responsibilities and ownership in collaborative BIM sharing. (Visit **www.consensusdocs.org** for more information).

Another IPD-related agreement was drafted by William A. Lichtig, a construction attorney with McDonough, Holland and Allen in Sacramento. The agreement is called "Integrated Form of Agreement" (IFOA) and is being used by Sutter Health on IPD health-care projects in California. The direct parties to the IFOA consist of the owner, architect, and GC and are known as the core group. This group is responsible for managing the process and makes decisions by consensus. Only when a consensus is not achievable does the decision default to the owner. Subcontractors are called "trade partners" and are selected from proposals and interviews by the existing IPD team members, including other trade partners already selected for the team. Selections are not made by price but by level of experience, and more importantly, by the ability to work within an integrated collaborative team. The IFOA provides for the use of Lean Construction tools such as Target Value Design, continuous improvement, last planner (one contract form of IPD), and "tightly coupling learning with action." (See "Integrated Agreement for Lean Project Delivery" in *Construction Lawyer*; it's posted at **www.leanconstruction.org**.)

Another organization, the American Institute of Architects (AIA), published two forms of IPD contracts last year. The first, a single-purpose entity, is a full IPD agreement with a limited liability corporation; all parties sign one agreement. The second contract is called multiple-purpose and has the more traditional separate owner-architect and owner-contractor agreements as well as a general conditions. (AIA sees this second form as an easier transition from DBB to IPD for a contractor experiencing IPD for the first time. Since it is structured similarly to traditional agreements, it will be more familiar to contractor that uses the DBB method.) AIA has also published *Integrated Project Delivery: A Guide*, which is available to everyone for a free download from AIA's web site. This document is an excellent overall guide to IPD. AIA's web site also has multiple informative Podcasts on IPD. (See **www.aia.org**.)

Finally, when public works accepts IPD, the conservative side of the business is signing on. In June of 2007, the state of Colorado passed a public law, 1342 "IPD Methods and Public Construction," meant to adopt the cost-saving benefits of IPD into public construction. The bill provides statutory authority to all public and quasi-public entities in Colorado to use any IPD methods deemed appropriate for the award of contracts for public projects.

Construction project delivery methods have come a long way. The latest, IPD, promotes true collaboration between the team players and gets the right people involved at the right time: the beginning. While it is still a relatively young approach, it is a viable and increasingly implemented option, and one that you should become familiar with in order to stay competitive in the business of construction. **MSC**

Dick Decker is a project manager with 25 years experience in construction management.

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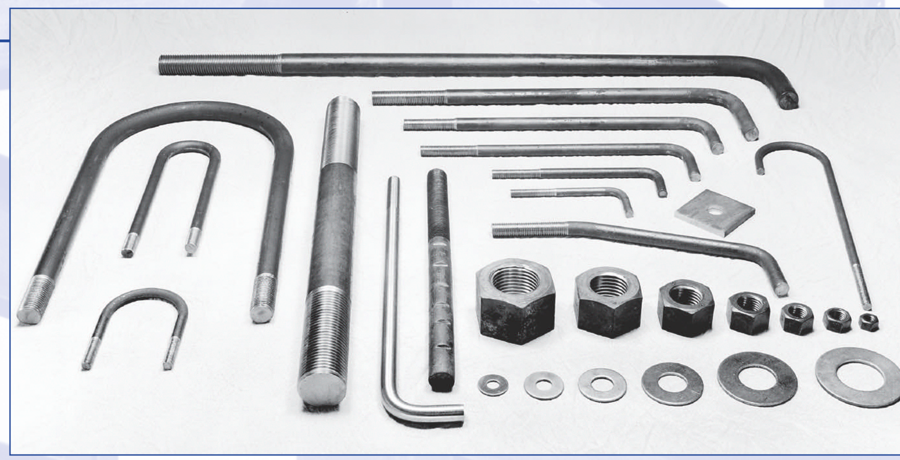

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Think Long-Term

BY JOHN P. CROSS, P.E.

The economy isn't out of the woods yet, but the design and construction industry can use this down time to its advantage for when building begins to grow again.

CONSTRUCTION STARTS IN THE FIRST QUARTER of 2009 for non-residential and multi-story residential buildings plunged to their lowest level in 40 years—which came as no surprise to building designers and contractors.

The trifecta of available financing, growing unemployment, and a reduction in both business and consumer spending for goods and services birthed another trifecta, that of a lack of new projects being designed, already-designed projects being placed on hold, and other projects being cancelled outright. During the last quarter of 2008 and the first quarter of 2009, many structural engineering and architectural firms either reduced staff sizes or transitioned to three- or four-day work weeks. AISC's Business Barometer, a quarterly survey of member fabricators, indicated a major increase in projects being place on hold or cancelled beginning in the second quarter of 2008 and peaking in the first quarter of 2009. All of this has left every design and construction professional pondering the short- and long-term outlook for building construction.

The Decline

The 45% reduction in overall building construction starts measured on a year-to-year basis is the result of three major economic events—a third trifecta, if you will. First, the general economy entered a recessionary period during the fourth quarter of 2007. This recession manifested itself in decreased consumer demand for goods and services and was accentuated by rapidly escalating fuel prices. Unemployment levels did not quickly accelerate upward, and construction starts were impacted at a rate consistent with moderate downturns of the past. This was in effect the "initial recession" and by itself would have resulted in approximately a 10% loss of building construction volume during 2008.

Second, the economy suffered a "credit collapse" that occurred at a time when the initial recessionary trend should have been bottoming out. Significant finger-pointing has occurred with respect to the ultimate cause of the credit collapse, but clearly issues surrounding the single-family housing market triggered the events of late 2008. An over-supply of single-family homes, falling housing prices, excessive and risky mortgages, and an overly complex system of risk-sharing derivatives all contributed to the severity of the collapse.

The credit collapse phase triggered significant decline in both consumer and business spending, a rapid increase in the unemployment rate, and a significant drop in the valuation of public corporations including some corporate bankruptcies. This third phase is in effect a "secondary recession," further decreasing the demand for non-residential buildings.

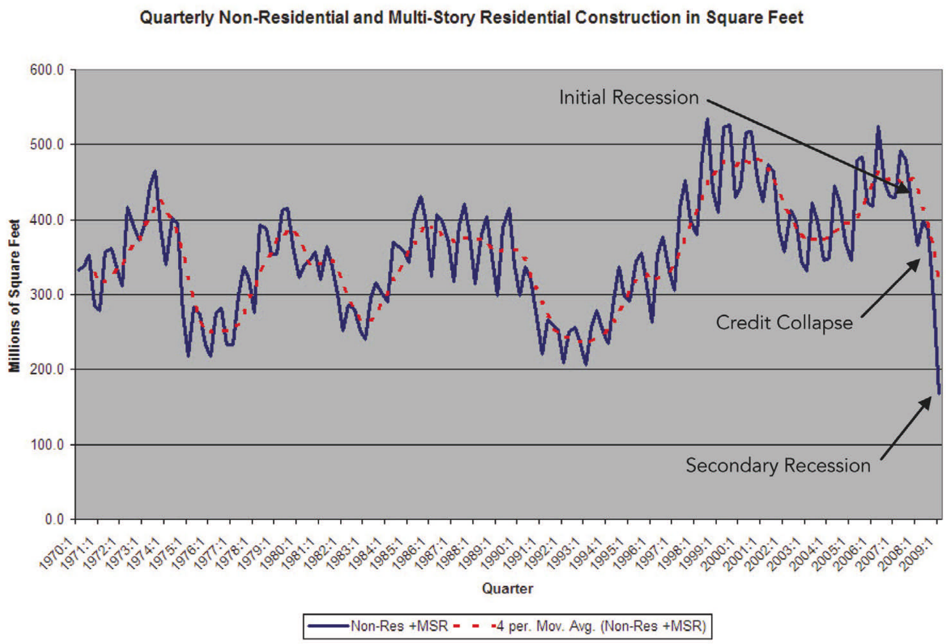
Construction volume will not increase significantly until the problems associated with each of these phases are resolved. Resolution will require addressing the underlying causes of the initial recession, increasing the availability of credit, and reversing the damage done to the economy by the secondary recession, although not necessarily in that order.

Available Credit

Before any recovery from the initial and secondary recession can occur there needs to be a perceived increase in the valuation of public companies and an increased availability of credit. This increased credit availability does not need to be accompanied by a loosening of credit standards, but rather simply an increase in the availability of funds to stable, qualified borrowers. During the spring of 2009 there has been an increase in valuation evidenced by a rebound in stock values, but as of early May the economy has yet to see an increase in the availability of funds from major financial institutions. An increase in the flow of funds into the credit market will probably occur in the third quarter of 2009 and signal the bottoming out of the recession. This increase in credit availability will not initially impact design and construction activity, but will support the continuing commercial business operations and result in positive GDP growth.

Even though the recession will be seen as bottoming out in the third quarter of 2009, national unemployment will not peak until early 2010 and may reach the 10% level at that time. Efforts to limit unemployment through the American Recovery and Reinvestment Act (ARRA) will have had a positive impact in keeping unemployment levels near or slightly above 10%, but will not result in a quick reduction in the unemployment rate.

John P. Cross is an AISC vice president.



The stimulus spending from ARRA will not significantly impact the building market. As of April 15, 2009, 3,072 projects had been accepted for funding under the ARRA program, using approximately 75% of the funding set aside for construction. Not surprisingly, 2,122 of those projects were for transportation infrastructure, 440 for buildings, and 510 for other infrastructure such as water and wastewater. Of the 440 buildings being funded, 372 were for alterations or interior work, four were additions, and only 64 were actual new structures. These new structures would include smaller structures such as visitor centers at national parks as well as some courthouses and office buildings.

Consumer and Business Spending

Typically, consumer and business spending will lead employment gains, and construction spending will lag employment gains. Increases in spending will translate into more jobs, but significant demand for new construction will not occur until the surplus of space resulting from the downturn is occupied. It is not anticipated that unemployment levels will return to pre-recession levels until late 2010 or early 2011. However, it will be important to monitor total employment levels rather than the unemployment rate, as total employment will drive the demand for new construction. This demand and the stabilization of the housing market should stimulate non-residential construction spending during 2011. Initial growth will probably be slow in mid-2011 but should accelerate rapidly into 2012.

It is important to recognize that unlike the single-family home market that was significantly overbuilt, non-residential construction was not in an over-supply condition at the start of the initial recession. The recession being experienced in the non-residential market is the result of a reduction in demand rather than a surplus of supply. Increases in demand at the end of a recessionary period occur more rapidly than the absorption of over-supply, which indicates that once the economy begins to expand, demand for new non-residential construction should ramp up sharply.

Weathering the Storm

The bottom line for the design and construction industry is that activity will not begin to pick up substantially until late 2010 or early 2011 and not return to 2007 and 2008 levels until 2013 or 2014.

During this recessionary period, there are positive steps that designers and builders can take to weather the storm and prepare for the future growth market. Today many successful designers and specialty contractors are approaching owners of projects that have been placed on hold with value-enhancing suggestions and firm price proposals that lower both project costs and the level of risk lenders are being asked to assume. At the same time, designers are integrating specialty contractors early into the design phase of a project to ensure design efficiency and maximum productivity.

Projects in smaller markets being funded by smaller, regional lenders are more likely to move forward with financing than are projects depending on large, national lenders or a combination of lenders. Schools, health-care, and governmental projects seem to have the best chance of moving forward in the current economic climate. Projects involving renovation work, particularly if related to energy-efficiency improvements, are also likely to move forward.

Green in Demand

On that note, projects that focus on sustainable construction will garner a larger share of the overall construction marketplace. Firms that can demonstrate expertise in the area of green design and construction as well demonstrate the implementation of sustainable practices in their operations will attract the majority of this work. Designers and specialty contractors with niche specialization will most likely continue to see a sustaining level of activity during this period.

Firms that resist the "I'm too busy in boom times but don't have the funds in lean times" mind-set and instead take the initiative to invest in new technology and the training required to efficiently implement that technology, will be well positioned to win the productivity battle in areas like building information modeling (BIM) and shop automation when a strong market returns. Similarly, firms that continue to invest in developing younger staff will be well prepared for the future.

Ready for the Recovery

There can be little question that these are challenging times for the design and construction industry. A significant turnaround will come, but it will not come quickly. The domestic economy has been significantly battered and will take time (and some intervention) to heal, but the long-term outlook remains positive. Population growth will continue to drive the demand for new non-residential construction. Replacement of older structures with new or renovated sustainable buildings will increase in pace. Domestic GDP will return to growth levels in excess of 3% per quarter, the typical threshold for significant construction activity. And the design and construction industry will emerge from the recession leaner and more efficient in addressing the demands of an expanding economy. **MSC**

Corrected Vision

BY DOUGLAS C. WOOD

The quality process must be seen clearly in order for it to work effectively.

MANY BUSINESS LEADERS TODAY understand the connection between process quality and business improvement. There are also many, however, who view quality as merely a form of risk management and who do not apply quality tools and personnel to business improvement initiatives.

We should care about this view because rapid improvement is essential to remain competitive and ensure a stable and sustainable business environment. Because of the easy international movement of work across borders we face three issues:

1. While the cash flow for single businesses may improve with the transfer of labor to another country, economic dislocations and cost to the overall society are not borne by the individual firm.
2. Political instability and associated risks to an extended supply chain are often ignored or denied, with business leaders hoping that political leaders will protect them from disruption. This is not assured.
3. We have all seen evidence of major mistakes made by business leaders in the last decade. These problems have often required government action to prevent economic disruption. The prevention of major economic disruption is laudable, but public funding of business mistakes cannot continue at the pace we have seen. There is simply not enough tax revenue.

Quality tools are about working smarter, and working smarter is the best way to achieve breakthroughs in efficiency and competitiveness. Proper application of quality tools is the best method to reach breakthroughs in competitiveness without raising the deeper risks of outsourcing. This is good advice that has been touted for decades. Why isn't the advice being applied?

Beliefs drive our understanding and reactions to the world around us. A belief (or myth) can help or hurt our decision making. To make faster progress, leaders need to identify and address myths that are beliefs based on false or inaccurate information.

"Blurred Vision," an article I wrote for the July 2008 issue of ASQ *Quality Progress* magazine, identified eight myths that get in the way of understanding quality's role in business improvement. Anecdotal evidence from many quality consultants and quality control specialists in a wide variety of industries shows that the majority of business leaders believe in some of these myths. The eight myths are presented here with a look at what may be misleading or false in each and what can be done to improve understanding.

Myth 1: Quality is strictly about product or service issues

Quality products are important, but to say that quality is limited to only products or services is just wrong. If you want to offer your organization's great products consistently *and* at the best cost *and* avoid being overrun by competition, you really do need organizational excellence. But what does that mean?

The statement by quality guru W. Edwards Deming, "It is not necessary to change. Survival is not mandatory" is as true as ever. What is really at stake here is not a just striving for excellence but survival. With companies around the world doing what the United States used to be uniquely good at—taking leadership in many areas of applied technology and business—survival for any firm is less assured than ever.

Organizational excellence in the United States is promoted by various state and national quality awards. You don't need to pursue an award to understand that these concepts outline what it takes to succeed. (You may download a free booklet that describes more about the core values and concepts from **www.quality.nist.gov/criteria.htm**.)

Your products and services will come and go, but improving the business process is how you build value that lasts. System-wide, comprehensive improvement is not easy and adoption of these core values and concepts does not guarantee success. The 'free' market has destroyed and will continue to destroy long established organizations. There is no perfect shield against business failure, nor is there a magic bullet for business transformation. Applying these core values and concepts will increase the probability of success.

Douglas C. Wood runs DC Wood Consulting and has worked in engineering and quality for 30 years.

Myth 2: Quality is only about controlling risk

This myth is not that good product quality controls risk (it does) but that many think that is all it does. Quality tools provide so much more than just

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risk management. Consider this: a carefully constructed product quality control program can reduce risk of quality defects with inspection, but no inspection program is perfect. A strong inspection program will be costly, and when the inevitable defects are passed (as always happens with inspection) your customer will be dissatisfied.

If your competition is getting better, or is about to become better, or a new competitor is entering the market, you are going to need real process control, not more inspection. You will need to develop a process that does not produce failures. This is where Six Sigma, Lean, statistical process control, and the many other quality tools really shine.

Myth 3: Cost-of-quality programs are old-school

According to this myth, measuring quality costs is an old concept no longer important. Fifty years old and known by various names—cost of quality, cost of poor quality, cost of poor execution, cost of conformance and nonconformance—many leaders feel there is little to be learned by applying this concept.

If quality is only about managing risk, you really don't want to track return on investment. Who considers the return on their insurance policy premiums? Quality control limited to risk management is like an insurance policy: your objective is to keep the premiums as low as possible and insure only what you need to insure.

To apply quality tools and methods that make your organization run like the proverbial Swiss watch, you need to use cost of quality as the key measure of all your improvement activities. This metric draws the connection between the prevention expenses and the costs of your mistakes, giving focus to improvement activities.

Myth 4: Quality is a discipline learned on the job, not in a classroom

Future business leaders need to learn quality tools and approaches from their master's level courses. Without teaching the quality body of knowledge, most of our future leaders will remain unschooled in what the quality tools can do for them, and unaware of the value that trained quality professionals can contribute to business improvement.

Myth 5: Six Sigma and Lean are great new tools

This myth speaks to use of tools. Six Sigma and Lean are great tool sets that use compilations of older proven tools, applied in better ways. Lean was created from Toyota's rethinking of Henry Ford's mass production process. Most of the tenets of Lean are industrial engineering techniques, to be applied by the normal production staff. Lean is industrial engineering for the masses, incorporating these tools from the existing body of knowledge for industrial engineering. Rewritten and used directly by workers, the implementation rate is much greater than when relying on a few engineers to apply the tools.

Six Sigma is quality engineering applied in a matrix of command and control than emphasizes project planning and management. Six Sigma is a powerful tool, but efforts can be wasted by working on processes that lack data, failing to use the Six Sigma command and control structure, and emphasizing statistical controls for processes that have significant special cause problems.

Certified quality professionals possess the knowledge to apply these tools effectively. Widely deployed improvements need to be applied by trained teams, with leaders that understand the tools well.

Myth 6: Choosing a quality approach is a task for senior leaders

Must senior leaders identify a quality approach? They *will* make the final decisions. What senior leaders may lack is an intimate knowledge of detailed business processes. The solution here is to make sure there is a robust upward communication flow within the organization.

Upward communication is difficult to sustain. Older leaders may view it as a distraction; new leaders as a threat. Tight command and control can stifle upward communication. Regular, thorough reviews of the processes of upward communication are needed to keep vital information flowing upward.

Myth 7: No preparation—besides willpower—is required to run an improvement program

When improvement approaches go wrong, it is often due to the idea that willpower will overcome poor preparation. Many firms skip good preparation, seeking quick results and fast action. It's not the fast action that's in question; rather, it's the confusion of action with progress.

Understanding current root problems, looking at data, planning, training, communication are all part of preparation that can make fixing a problem or making an improvement change quick and straightforward. Lack of good preparation often results in fixing symptoms or addressing the wrong issues.

Costs are a second excuse to skip good preparation. Training is expensive, and the better the training, the more it costs. You can buy a business help book, skim it, try to apply what it tells you, and even succeed some of the time. Getting a group to all understand the same ideas and commit to working together as a team does not happen in a book. Good preparation can take weeks to months, and there will be little short term return for the investment in preparation.

Myth 8: Cost, quality, and schedule form an iron triangle

This myth says improvements in these three areas are mutually exclusive. To improve any one, you will hurt one or both of the others. For example, to make a shipment on schedule, either product quality is sacrificed to speed up the process, or significant labor additions (higher cost) are needed.

If quality is seen as risk management, the above example seems intuitive. It is not true if the focus of quality is on both process *and* product. If the quality of a process is improved, it will run faster, produce fewer defects and cost less. In other words, process quality underlies both service and cost.

Survey

Evaluation of the myths that we all use to process our view of the world is sometimes needed to make forward progress. With some effort, we can avoid the limitations that these myths place on our drive toward improvement. And be sure to take the survey on "8 Quality Myths" at **www.dcwoodconsulting.com**. Register your e-mail at the survey to receive study results. MSC

Douglas C. Wood's book is The Executive Guide to Understanding and Implementing Quality Cost Programs. *He is certified with ASQ as a CQE and CMoQ/OE and is a member of PMI. He has worked with multiple state quality awards and is currently the chair of the Quality in Program Management committee of ASQ's Quality Management Division. Visit* ***www.dcwoodconsulting.com*** *or call 913.669.4173.*

Changing of the Guard: How to Avoid a Leadership Shortfall

BY JOSEPH D. REI, PH.D., AND F. LEIGH BRANHAM

Identifying the next generation of leaders is a significant issue for current design firm leadership.

THE FUTURE OF THE A/E INDUSTRY and the firms that design our built environment is dubious. The United States and our global society have clear needs, but doubt exists about who will lead the transitions facing the firms and the industry. Increasingly, transitions in the way projects are delivered, in the roles played by design professionals, in the manner in which new professionals are recruited and retained all demand increasingly skilled leadership at a time when the industry's leadership talent pool is decreasing. At risk is the future of these professions.

The Role of Teams

Design (A/E) firms are project-based businesses that are continuously forming and reforming project teams. The generational research suggests team-based, project-focused firms may be impacted more than other types of firms. The success of a project depends on moving the team to a high level of performance in a relatively short period. When teams contain members from different generations, a series of assumptions and unstated expectations often influence performance. The old adage of "form-storm-norm-perform" that describes team performance should be closely monitored to ensure the firm builds a core competency of team building and team performance.

The popular press has addressed "who" will lead the way in a multitude of forums. The concepts of multiple generations in the workforce and the eventual retirement of the baby boomers are documented thoroughly. Some are enlightened by the research on generations, while others are rankled by the way the generations are stereotyped. Regardless of your take, there is no denying that leaders of design firms are facing significant challenges and workforce issues.

There is an ever-increasing set of complexities that CEOs and firm leaders face regarding how design firms deliver their services and how projects are created and completed. In the May 2008 issue of *Dwell Magazine*, Steve Silberman provided a summary of the *AIA International Committee Report on the Offshore Outsourcing Roundtable (2006)*. In 2006, U.S. architectural firms outsourced 69% of all construction documents, with 9% going overseas. That number is expected to continue increasing, and the AIA Committee estimated that by 2015, 20% to 30% of U.S. architecture jobs would be offshore.

Sweeping Changes

Sweeping changes over the last 30 years such as increasing divorce rates, single parenthood, and global competition have greatly shaped the development of different generations. Experts argue that the proliferation of video games and computer access have had their effect on the brain conditioning and personality development of large numbers of Gen-Xers and millennials. Such stereotypical perceptions, even if inaccurate, are part of the reality that firm leaders must deal with daily.

Meanwhile, many millennials and Gen-Xers, who have watched parents lose their jobs after lifetimes of sacrifice for the good of the company, cannot grasp why they should be loyal to a heartless corporation. They seek a "new deal"—since they feel they cannot realistically expect lifetime employment. Instead they expect "lifetime employability," which to them means challenging assignments and lots of learning. With the looming talent shortage, talented younger workers know they have other employment options and now insist on better management, including more frequent feedback and more flexibility about where and when they work.

Polarization

In many workplaces we see a polarization of generations that erodes the trust necessary in developing both current and future leadership for the firm. Both older and younger workers have taken a "we're right, you're wrong" position. This shortsighted stance is not helpful in building an environment that will nurture the next generation of design firm leaders. This, to say the least, is not a recipe for an effective leadership development program.

The first and most critical step

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How the Generations Differ in Attitudes and Values

Large numbers of traditionalists (now age 62+)
- ➜ Lack technological skills
- ➜ Value loyalty, compliance and dues-paying
- ➜ Expect younger generations to value what they value
- ➜ Believe their way is the right or only way
- ➜ Are disengaging and preparing for retirement

Large numbers of boomers (now ages 42 to 61)
- ➜ Limited in their technology abilities
- ➜ By their mere presence are blocking the upward advancement of many Gen-Xers
- ➜ Believe "if you train 'em, they'll just leave"
- ➜ Believe in and practice "hands-off" management
- ➜ Have sacrificed family and work-life balance for career advancement
- ➜ Believe many Gen-Xers and millennials lack a work ethic
- ➜ Expect younger generations to value what they value

Large numbers of Gen-Xers (now ages 27 to 41)
- ➜ High-tech skills
- ➜ Are frustrated with limited promotional opportunities
- ➜ Feel more loyalty to their own careers than to the organization and have little reluctance to change jobs
- ➜ Think about self-employment as a desirable option
- ➜ Believe it doesn't matter when and where they work as long as they get the job done
- ➜ Want to have rich personal and family lives outside of work
- ➜ Are impatient with "unrealistic expectations" of millennials

Large numbers of millennials (now 26 and under)
- ➜ Eat/Breathe/Sleep technology —they are so skilled, it is invisible to them
- ➜ Have received intense parental attention, structure, feedback and coaching
- ➜ Expect to receive the same from their managers in the workplace
- ➜ Believe they are special and deserving of praise and recognition
- ➜ Expect their jobs to be challenging and meaningful
- ➜ Want to fit a variety of activities besides work into their workweek
- ➜ Believe it doesn't matter when and where they work as long as they get the job done
- ➜ Recognize that they have other employment options and do not hesitate to move on when things turn sour

is to recognize several undeniable facts. The first is the changing composition of the workforce. With 78 million boomers on the retirement threshold, only 44 million Gen-Xers in line to replace them and most millennials just entering the workforce from college and lacking the experience to move up, the workplace will look very different in 10 years. How will firms deal with the coming talent and leadership shortages, especially if the economy turns around after the presidential election and reaches previous growth rates?

Leaders with foresight and longer-term perspectives are investing in succession management, creative recruiting, and employee retention and engagement initiatives now, so when the leadership talent crunch increases as 2010 approaches, their firms will be far ahead of their competition.

Immediate Concerns Facing Leaders

The more immediate challenge for current leaders is devising how to develop future leaders who may have markedly different attitudes about life-work balance, teamwork and company loyalty. Progressive leaders must start by acknowledging that they must understand the strengths of each generation and work to develop those strengths, as well as finding ways to affect essential knowledge transfers from the firm's senior leaders to its future leaders.

Some firms now recognize that they no longer have the luxury of waiting for a good prospective leader to develop for five or 10 years before the firm expresses an interest. Gen-Xers and millennials will not wait long to be tapped for future leadership roles, and they are willing to jump ship to firms that promise them immediate responsibility and advancement. Forward-thinking leaders must engage promising future leaders early, develop them aggressively and provide them with the mentoring that will bring them up to speed in a shorter time frame than tradition dictates.

Immediate Actions for Leaders

How can you accommodate differences in approach when this feels akin to compromising your firm's long-held values?

Members of each generation have earned the right to see the world in their unique way. Not all members of each generation share the same world view, but each generation deserves to have its world view understood and respected. Organizations and leaders who don't accommodate these diverse needs and expectations, and attempt to impose a uniformity of values, will fail to attract, engage and retain the talent their firms will need to succeed.

The Advanced Management Institute for Architecture and Engineering (AMI) recently conducted a survey with 155 managers and staff of several professional services firms. One of the questions from the survey was "What should we do about intergenerational challenges?" The following are the responses:

- ➜ 34% Train all employees to understand and accept generational differences
- ➜ 27% Increase coaching of younger generations
- ➜ 22% Train management how to manage and motivate other generations
- ➜ 7% Perform more selective hiring
- ➜ 6% Do nothing at this time
- ➜ 2% Conduct selective terminations
- ➜ 2% Other: Hold more social events, use mixed teams, etc.

We wonder about the 6% who advocate doing nothing. Perhaps the inter-generational teamwork and communication at some firms is so smooth and clear that no more work needs to be done. Some firms still have so few millennials on staff that the full force of changing expectations has not yet struck home. That said, doing nothing is exactly the wrong way to go.

AMI followed its research with a focus group to help establish best practices for developing the future generations of leaders. The following ideas emerged and, in most cases, are being used to great advantage.

Self-Awareness

- ➜ Leaders must be self-aware. This self-awareness is a precursor to authenticity. Help them become authentic and understand others by using assessment instruments and providing the opportunity to discuss the results.
- ➜ Use competency inventories and personality assessments to increase employee self-awareness.
- ➜ Share this information within the organizational work teams so that all generations and organizational levels have a better understanding of their team(s) and team members.

Training and Development

- ➜ Allow interns to design their own Individual Development Plans.
- ➜ Provide significant training and development opportunities. Young workers no longer expect lifetime employment, but do value lifetime employability. (Paradoxically, they are more likely to stay with employers who make them more widely employable.)
- ➜ Allow interns to propose their own training topics, and invite senior staff to attend and participate.
- ➜ Train all employees in fundamentals of quality control and improvement, diversity, communication and leadership so that all generations speak a common language.
- ➜ Teach the principles of change management and how to overcome resistance to change. Show all employees why logic does not always convince the minds of resisters.
- ➜ Teach listening skills and negotiations as a key aspect of change management.

Coaching and Mentoring

- ➜ Allow younger workers to choose or gravitate to preferred mentors.
- ➜ Train mentors in coaching and mentoring skills before pairing them with mentees.
- ➜ Ask younger employees to provide "reverse mentoring" to older staff and encourage older staff to seek such mentoring.
- ➜ Consider group mentoring, where one senior leader meets periodically with multiple participants. This can be a useful option for transferring knowledge of firm history, best practices, client and project case studies and similar knowledge carried by senior leaders.
- ➜ Identify early adopters of newer technologies among older generations—they can relate to their peers and communicate with them more effectively than younger staff. These early adopters may also help bridge communications.

Ownership and Leadership Transition

The small size of the Generation X cohort may challenge owner transition. In the American workforce, Generation X is the smallest of all the current cohorts and will not be increasing in size. For any specific design firm, this may not be an issue. However, taken as a whole, in 10 years fewer potential purchasers and leaders will be around when the boomers are ready to exit.

This has several potential impacts: Will your firm have a full complement of internal buyers with enough wealth to purchase the shares as priced? Will you have to sell shares to employees who are younger than what you would like? How are you going to prepare them to lead the firm? When making a transition, we recommend separating the ownership issues from the leadership issues. But even that ends in the same questions: Will there be enough? Will they be ready? What will be the strategy to get them ready for the transition?

Filling the Leadership Pipeline

The vitality of architectural and engineering firms depends on recruiting, hiring, developing, and promoting a steady supply of great staff and employees. The different expectations and needs represented in the multiple generations increase the challenge to this all-important process. The human capital challenges facing senior leaders and managers in design firms should not be taken lightly. Senior managers are being challenged to adapt their managerial style to ensure that individuals and teams perform at their highest levels. Managers and leaders often feel like they are spending too much time in this arena, but there could be no more important job in the firm. The talent of these generations is evident, the differences in working styles are pronounced, and the manager's job is to tap into these sources of energy to produce the highest quality services. MSC

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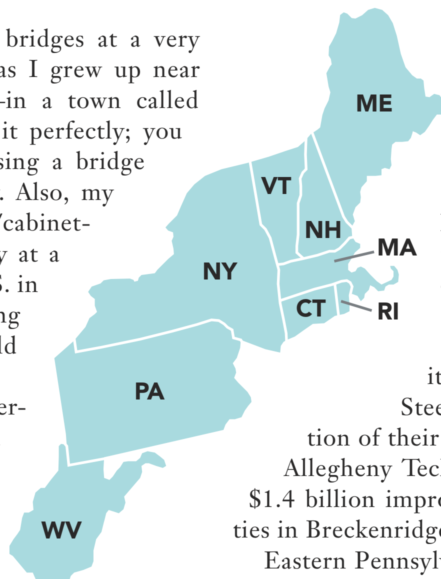
From the Heart of the Steeler Nation

BY BILL PASCOLI

From a very young age, it seemed that fate was driving AISC's New England regional engineer toward a career in steel.

I DEVELOPED AN INTEREST in bridges at a very young age. Perhaps it was destiny, as I grew up near America's steel capital, Pittsburgh—in a town called Bridgeville. And its name describes it perfectly; you can't leave Bridgeville without crossing a bridge over Chartiers Creek or a tributary. Also, my father was a self-employed carpenter/cabinet-maker who learned his trade in Italy at a young age and immigrated to the U.S. in 1929. When I was 14 he began teaching me how to work with tools and build things.



In my junior year as a civil engineering student at Penn State, I passed by the American Bridge Division (ABD) of the U.S. Steel fabrication plant in Ambridge, Pa. On the roof of one of the buildings was a sign that claimed that the Ambridge plant was the largest steel fabricating plant in the world.

I eventually went to work for ABD and learned just about everything I know about structural steel during my 20-year career there. Probably the most important lessons that I learned were that 1) *people* build the buildings or bridges and 2) the fabrication and erection processes involve the same amount of structural engineering as the actual design of the structure.

I started my career at AISC in 1996 as the regional engineer for the Upper Midwest region. In 2002 I became AISC's national project director for the parking garage and marketing-to-decision-makers initiatives, then returned to the field in 2005, this time as the regional engineer for New England. My territory includes Maine, Vermont, New Hampshire, Massachusetts, Rhode Island, Connecticut, Pennsylvania, and West Virginia, as well as upstate New York from Buffalo to Albany and approximately Poughkeepsie to Plattsburgh and Gouverneur. This is a lot of geography to cover, but there are a number of major metropolitan areas that provide a gauge for what is going in the region.

What's Happening Back East

In Pittsburgh our young mayor, Luke Ravenstahl, claims that the city is currently going through its third renaissance, with the new Rivers Casino along the Ohio River, which is scheduled to open in the fall, and the Pittsburgh Penguins' new hockey arena, which will open in time for the 2010 season. These two steel-framed projects and the LEED-Certified David L. Lawrence Convention Center have stimulated the construction of several steel hotels, condos, and mixed-use buildings in the downtown area. There is even some activity in the industrial sector, with U.S. Steel planning a $1 billion modernization of their coke-making facility in Clairton, Pa. Allegheny Technologies has also made plans for a $1.4 billion improvement to their steel making facilities in Breckenridge, Pa.

Eastern Pennsylvania is showing signs of growth too. The major expansion of the steel-framed convention center in downtown Philadelphia is stimulating the construction of hotels and mixed-use buildings in the short term and possibly several high-rise buildings in the long term. Two casino projects in the area that were licensed the same time as Rivers Casino in Pittsburgh have not yet broken ground due to local and city objections to their locations along the Delaware River. However, the state of Pennsylvania has recently drafted legislation to encourage the city to reach an agreement with the casino licensees to allow them to build or find new sites for their facilities. Time will tell. Another (steel) casino project, the Sands Bethworks Casino in Bethlehem, Pa, which is scheduled to open in the fall, is located on the site of the former Bethlehem Steel facility.

Boston is undergoing a renaissance of its own, with the emergence of the Rose Kennedy Greenway, which is the space reclaimed after the elevated central artery structure was removed as a result of the "Big Dig" project. The city is now reconnected and development is taking place in the Financial and Seaport

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Districts, with a new convention center and hotel, office, and residential buildings. More development is planned, including several steel high-rise office and mixed-use buildings in tandem with the health-care and life-science construction in Cambridge and Boston, which continues to be an important part of the city's progress.

Waterfront development has been the key to the economic development of upstate New York, but the loss of revenue has crippled the state's plans for most projects, including the new convention center in Albany and the renewal of downtown Buffalo. It is encouraging, however, that developers continue to invest in property for future consideration in the corridor from Buffalo to Albany.

West Virginia, while small, it is not insignificant, since the coal industry is still such an important factor in its economy. I hope to support the fabricators in this state and help grow the market for steel in other sectors including parking, office, institutional, and residential.

The Fabrication Picture

During the fourth quarter of 2008 and the first quarter of 2009, I visited more than half of the AISC member fabricators in my region and plan to visit the remainder during the second quarter of 2009. In general most fabricators are optimistic and have a strategy to survive the current recession. Many of them will be making those long-planned improvements in the shop or completing the process to become AISC Certified in the bridge, bridge component, or building categories.

There are four regional fabricator groups spread throughout my region. The Steel Fabricators of New England (SFNE) has reinvented itself and is currently very active with internal membership meetings on current topics and an education committee tasked to provide programs for their members and outreach to the AEC community. Recent meeting topics have included the application of the theory of constraints to running a successful business and the group's 28th annual engineers' conference at the Worcester Polytechnic Institute (visit **www.ssfne.org**).

The New York State Steel Fabricator's Association is also very active in upstate New York and usually has two membership meetings per year that feature a current topic along with an open exchange of information on the state of the business in the area (visit **www.nyssfa.org**).

Currently, the fabricator group in West Virginia and the Mid-Atlantic Fabricators Association—which covers central and eastern Pennsylvania, New Jersey, and Delaware—are relatively inactive but hopefully will be revitalized with the new Regional Fabricators Group Summit that Carly Moore, AISC's membership director, is fostering.

On Parking

I classify myself as the "quixotic" champion for steel-framed parking structures at AISC, since I have been involved in the initiative for nearly eight years. To promote the use of steel as a viable material for parking garages, I represented AISC at the International Parking Institute's conference, which took place earlier this spring in Denver, and will also be at the National Parking Association's conference in the Washington, D.C. area, October 11-15. On a regional basis I will also be participating in parking association programs in Pennsylvania, New York, and New England.

Besides being home to the Steeler Nation (which is still ecstatic after the team, only months ago, won its sixth Super Bowl trophy!), Pittsburgh is also where you will find the best example of steel-framed parking garages that have withstood the test of time. The garages at Station Square, Duquesne University, Carnegie Mellon University, and Allegheny General Hospital, to name a few, are all still serviceable. If you are ever in the area, I would be happy to take you on a tour. MSC

The Nuts & Bolts of a Building—Literally

BY THOMAS J. SCHLAFLY, MONICA STOCKMANN, AND GEOFF WEISENBERGER

Photos: Geoff Weisenberger

Shot-blasted nuts.

Steel arrives in bar form directly from the mills.

The ends of the bars are color-coded by grade.

WHEN IT COMES TO STEEL-FRAMED buildings the beams and columns, due to their high visibility, get all the glory. But a building is certainly more than what meets the eye. In particular, bolts are a very necessary but often unnoticed component in many buildings. Although bolts are typically unseen in the finished building, bolt quality is of utmost importance to the building's safety.

Questioning Quality

On the subject of bolts and safety, last year AISC received several questions regarding defective bolts. A few disturbing pictures were circulating around the steel construction industry, showing structural bolts that appeared to be bleeding; seams on the bolts were highlighted with dye penetrant.

Many people expressed concern about these bolts and asked if there was a problem with the bolt manufacturing process as a whole. AISC looked into the matter and concluded that the pictured bolts did not indicate a systemic problem in the manufacturing process, and responded that the ASTM bolt standards define defects, acceptance criteria, and sampling plans. These ASTM quality elements, along with reliance on a fastener manufacturer that is quality certified as required by the Fastener Quality Act, result in reliable fasteners in which a project team can have confidence. That does not mean that there will never be defects, but it does mean that they will not be frequent, systemic, or in quantities that would compromise typical structures.

Nuts, still orange-hot, after being formed.

Punch-outs (left, in photo) are recycled.

Here at AISC, we found one of the ASTM quality elements especially intriguing: The quality sampling plan looks for defects on a statistical basis. (This is sufficient if the producer's manufacturing processes result in consistent quality.) This inspired us to learn more about the fastener manufacturing process, the typical quality-related issues encountered, and the quality controls a good manufacturer uses to provide the consistent level of quality the industry relies on.

Trekking to Peru (Illinois)

Fortunately, Unytite, a large bolt and nut manufacturer, is located just 120 miles south of the AISC offices in Peru, Ill. Chuck Hundley, the company's operations manager and a member of the Research Council on Structural Connections, invited a few AISC staff to visit his facility this past spring to observe the bolt and nut manufacturing process and to discuss the quality checks that occur to make bolts safe products for use in structures.

Hardening involves temps of up to 1,570 °F.

Unytite may not be well known to fabricators and engineers, because they sell fasteners through distributors rather than directly to fabricators. But this allows them to focus their resources on manufacturing while the distributors provide customer service to the end users. Unytite, a family-owned company, was established in Kobe, Japan, and its Peru plant has been in operation since 1990. Roughly 43% of the facility's output is structural bolts, 21% is structural nuts, and the remaining 36% is for automotive and OEM applications.

In the structural market, Unytite has the capacity to manufacture both tension-control (TC) and standard bolts, ranging from ⅝ in. to 1¼ in. diameter in size; and nuts ranging from ⅝ in. to 1⅝ in. diameter. Its structural fastener offerings include ASTM A325TC (F1852) and A490TC (F2280) structural bolt/nut/washer assemblies; ASTM A325 and A490 hex head structural bolts; ASTM A194 Grade 2H heavy hex nuts; and ASTM A563 Grade DH and DH3 heavy-hex nuts. Their Japan plant can provide fasteners up to 48 mm (1⅞ in.) in diameter, 1,000 mm (40 in.) long.

Bolt-threading equipment.

Threaded bolts on the move.

On the shop tour, we observed the manufacturing of nuts, structural bolts, and other fasteners. We started with the raw materials. The pictures that we saw last year were of bolts manufactured outside the U.S. and not by Unytite in Illinois or in Japan. We could see a manufacturer's mark on the bolt heads but could not find that mark in the register of known manufacturers. Wherever they were made, it is very likely those seams were a result of a flaw in the raw material.

In Unytite's plant we saw several coils (called wire for cold forming and bars for hot forming) of steel; coils range from ½ in. to 1¼ in. and bars range from ⅞ in. to 2 in. in diameter. Hundley explained the lengths to which Unytite goes to qualify suppliers. They order material that is similar to ASTM standards but modified to meet their needs, and the steel is straightened and eddy current tested for surface discontinuities (seams) under very strict tolerances. Different grades of steel in the stock piles are indicated by different colors at the ends of the bar.

Nuts

Nuts are produced via hot forming. A bundle of steel is loaded on a bar rack system that automatically feeds the steel into the induction system. The steel is heated to a bright orange color (around 2,300 °F) in an inductance heating coil, cut to short lengths, and forced into a hex die, and then a plain unthreaded hole is punched. Depending on the size and complexity of the product, this operation occurs at a rate of about 90 to 175 nuts per hour. The nuts are marked in the forming process, where the punch has the manufacturer's mark and identifying grade. The punch-outs are diverted away from the nuts and are eventually recycled.

The next process is to clean the scale off of the nut. The nuts are tumbled in a rotary drum with steel shot (roughly the size of sand), and the shot effectively removes scale from the nuts. The nuts are then heat-treated (and cooled with water) to obtain the specified hardness.

The nuts are then threaded or "tapped" on a bent shank-style tap that is about a foot long. One nut pushes the next over the teeth of the die, and the nuts at the end fall off of the die as the next nut is pushed on. Unytite has 96 spindles to perform tapping. Automated equipment feeds the nuts to various machines, and each machine has either two or four taps.

Fit of the thread is an important quality criterion Unytite has to control, and wear of the thread-cutting taps is a main concern. Unytite employs pre-set counters that keep track of how many nuts the tap can thread before re-sharpening is required. Additionally, to control this wear and the thread fit, the technicians operating the threading machines routinely test the nut threads with a "go/no-go gauge." Nuts that are hot-dip galvanized are threaded after the galvanizing has been applied, while nuts that are mechanically galvanized are threaded before.

After the tapping, nuts used for TC bolts, as well as galvanized nuts, are lubricated. Lubrication is one of the most important facets of TC bolt performance. Improper lubrication can result in either high pretension and broken bolts, or low pretension. Lubricants are proprietary products and their composition is typically a valued trade secret for a bolt manufacturer. Unytite's lubricant was selected for its slip coefficient, consistency, and ability to remain consistent through reasonable storage and construction conditions.

Lubricants are usually colored (for coated products) so that the manufacturer and user can see that the nuts are lubricated. It was of interest to learn that the surfaces that require lubricant are both the thread *and* the bearing surface of the nut. The bearing surface often provides more resistance to torque than the thread.

Lubricant application is similar to a plating line for electo-plating. There is a predetermined weight that will be in each basket. The baskets are transferred to heated tanks that have cleaners and chemicals to coat the nuts. Each tank is monitored for the concentration of the chemicals. The tumbling action of the baskets ensures proper cleaning and coating for the entire nut. Coating of the bearing surface and threads is the key to controlling the torque coefficiency for the assembly.

Bolts

While nuts are hot formed, structural bolts are cold formed using wire that has already been cleaned, coated, drawn to size, and annealed if necessary. A bolt is formed in stages—which all occur in one forge—transferred through a series stations with various punches and dies that form the part into the specified product. Simply put, the punch, which is indented in the shape of the rounded head, strikes the bar and creates the head (and applies manufacturer and grade markings). Then the bolt is sheared to the desired length and the shank and threaded areas are formed. The process happened so fast that we could not even see the bolts move from one stage to the next. Most of the structural bolts Unytite makes have round heads. Hex-head bolts are made by trimming the sides off of round-head bolts, and these trimmings are also collected and recycled. TC bolts undergo an additional process; a 12-point spline/pintail is added to the end opposite the head. Average output for structural bolt forming is around 100 pieces per minute.

The blank unthreaded bolts are then threaded using a rolling process, as opposed to the tapping process used for the nuts. Rolled threads are formed by inelastically pushing material from the thread root to the thread tip. Cut threads, as the name implies, are formed by removing material from the unthreaded shank of the bolt. The threading machine is fundamentally a ring, about 15 in. in diameter, and a disk, approximately 13 in. in diameter, positioned such that the space between them becomes slightly narrower as the bolt progresses through the forms. Both the ring and the disk have grooves to form the threads. The disk turns relative to the ring so that the bolt rolls through the space between the thread forms. The notch diameters for TC bolts are also formed during the thread-rolling process. As one would expect, the tolerances are very tight and the forms wear over time—so as with nuts, thread fit for bolts is a quality control point that Unytite watches closely.

Tapping machinery threads the nuts.

Bolt assemblies being packaged for shipping.

Because Unytite's cold-forming processes do not produce scale, shot-blasting of the bolts is not required as it is with nuts. Thus, the heat treatment and quench hardening process is the next step for the bolts. Unytite uses a continuous mesh belt with a 2,200-lb/hour capacity, as quality control is easier with smaller batches (if 2,200 lb per hour sounds like a lot for something called a "small batch," know that some furnaces can process nearly twice that much).

Heat treatment is certainly a closely timed, precision process, as raising or lowering the temperature of the bolts too quickly can create significant differences in appearance and quality. The bolts are heated to about 1,570 °F, quenched in oil, and then tempered (around 900 °F) to the specified hardness range. The appropriate temperatures are dependent on the type of material and the specified hardness. Unytite prefers to perform the quench and temper process in-house for cost and quality control reasons. Bolts do not require lubrication, so once the bolts cool, they are ready for assembly and/or packaging.

TC bolts, nuts, and washers are shipped in an assembly to help control thread fit and installation pretension. Unytite buys the washers from an outside supplier, and the nuts are turned onto the bolts automatically. The assembly machines work at around 40 to 45 assemblies per minute, although they are capable of 50. The assemblies are dropped in kegs, which are marked with the specified grade, size, and assembly lot identification and then palletized. And it's fair to say that the majority of the nuts and bolts make it into the kegs. The nut manufacturing process results in about 8% scrap metal (the punch-outs), and less than 1% of finished bolts becomes scrap (the shavings from the trimming and deburring operations).

The Lab

The last stop in our tour was the bolt testing laboratory. Think of it as a high-tech crime lab with the victim being a bolt or nut and the crime being anything from a crack to unacceptable hardness and tension levels. One machine is a "Super Skidmore" of sorts, a larger version of the Skidmore machine that tests bolt tension. Another machine, which performs magnetic particle testing, uses a black-light-sensitive powder, applied to the bolt's surface, to bring cracks "to light." There's also a hardness testing machine. Other testing devices/procedures for structural nuts and bolts include calipers, micrometers, functional gages (go/no-go), and tensile and proof load testing. The lab performs the tests required by the ASTM standard using the specified sampling plan.


Bolts on display in the testing lab.

A hardness testing machine in the testing lab.

The QA sampling plan provides certification that the product conforms to the specification. But this only works if production is in control. That in turn is done with many QC tests, vendor controls, and process checkpoints performed on the floor throughout the manufacturing process.

While nuts and bolts are clearly tiny in the scope of the entire framing system, the sheer number of machines and processes—as well as the quality control—that go into manufacturing such small items is large. And this is a testament to the equally large role bolt assemblies play in bringing—and holding—buildings together. MSC

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Controlling Tension

Structural bolts became a prevalent method of connecting steel in structural frames in the late 1950s and early 1960s. Tension-control (TC) bolts were introduced in the 1970s and have become the preferred fastener in building construction. TC bolts have a reduced neck or notch between the spline and the end of the bolt. The area of the material at the notch controls the torque and tension of the bolt when the spline shears off during installation. In pretensioned connections, TC bolts provide an indication that the required tension has been reached in the installed bolt. They also permit the use of an electric wrench, because the nut is turned against the bolt shaft, not against the resistance of the person holding the wrench.


TC bolt assemblies, ready to be shipped.

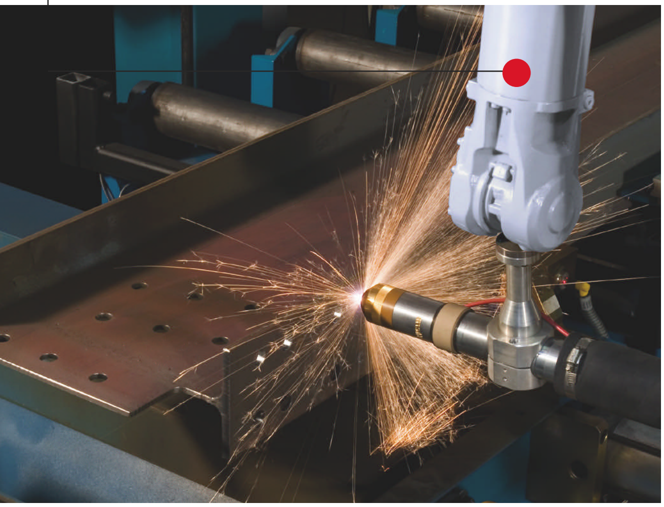
Each month MSC's product section features items from all areas of the steel construction industry.

In general, these products have been introduced within the past six months. If you're looking for a specific product, visit MSC's online product directory at **www.modernsteel.com/products**. You can browse by product category or search on any term to help find the products you need, fast.

A Productive Ten Minutes

The PythonX Structural Fabrication System from Burlington Automation is designed to use *all* the features available from detailing software—not just some of the data. This includes coping data, bevel angles, layout marks, bolt hole positions, reference points, and part marking. A timed demonstration of the complete fabrication of an 8-ft W16x31 structural beam, using the PythonX CNC system, was recently videotaped and released to the public. The video can be streamed from the "Economics" page of the PythonX web site, **www.pythonx.com**. In the video, PythonX fabricates the following features in a single pass:

- Front trim miter cut ¼ in.
- 6 bolt hole angled cluster on web
- 2 bolt holes on bottom flange
- Piece mark—2 Lines—9 letters total
- 2 copes on the front of the beam
- 4 layout marks on web
- 3 slots on web at rear of beam
- Notch cut on flange
- 3 bolt holes at front of beam
- 2 bolt holes on top flange
- Cope on rear of beam
- Flange notch cut flush with web (both sides)



Total processing time: 10 minutes, 13 seconds.

No programming of the cuts was required. Structural steel fabricators who examined the beam design suggested that performing this fabrication using traditional manual methods would take roughly 120 minutes. Using a CNC drill line and bandsaw, plus manual torch for the copes and flange flush cuts, would typically take approximately 82 minutes (this does not include time for moving the beam between operations). Therefore, PythonX completed the beam fabrication in less than 12% of the time expected for a shop using conventional automated equipment.

For more information please visit www.pythonx.com or call 905.689.7771.

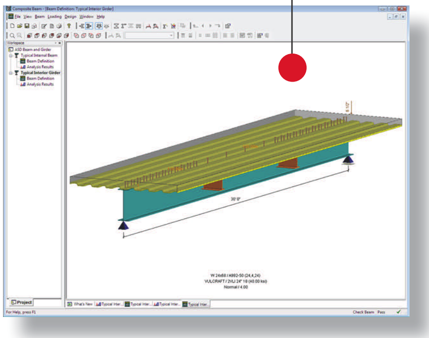
Take a Test Drive

FASTRAK Building Designer, available from CSC, Inc. equips structural engineers with a tool to model and design buildings quickly and simply within a user interface suitable for any building geometry. As an introduction to this software, CSC is offering the FASTRAK composite beam module to every U.S. structural engineer completely free and without obligation. Key benefits of the module include:

- No cost; unlimited licenses per company
- Free access to training videos
- Tutorials, worked-examples, and sample beam files included

Key capabilities of the software include:

- Composite/non-composite beam design
- Comprehensive and customizable output
- Optimization of beam based on user criteria
- 2005 *Steel Specification* (ASD/LRFD)



For more information please visit www.cscworld.com or call 877.710.2053.

All products submitted are considered for publication, and we encourage submittals related to all segments of the steel industry: engineering, detailing, fabrication, and erection. Submit product information via e-mail to Geoff Weisenberger (weisenberger@modernsteel.com). To be included in MSC's online products directory, contact Louis Gurthet (gurthet@modernsteel.com).

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Don't Force It: Roam Before You Pave

BY PAUL WILLIAMS

Sometimes it's best to let natural progression take precedence over conventional wisdom.

I HAVE A STORY FOR YOU. An architect built a cluster of office buildings around a central green. When construction was completed, the landscape crew asked him where he wanted the sidewalks.

"Just plant the grass solidly between the buildings," was his reply.

By late summer, the new lawn was laced with paths of trodden grass between the buildings. These paths turned in easy curves and were sized according to traffic flow.

In the fall, the architect simply paved the paths. Not only were the paths beautiful, they responded directly to user needs.

I love this story.

I am certain this unconventional approach caused the building owners to think the architect was crazy. Nevertheless, with patience they came to see the brilliance of his approach.

Take it Easy

Think about it: Are there things you are forcing? Places where you should ease up?

The lesson for me is "roam before you pave." If sidewalks were immediately installed, they would have had sharp, rigid angles and uniform width similar to every other sidewalk. However, by allowing them to form organically, it put the right-sized path in the right place.

Applying this lesson to business, why not let customers test your prototype product, service, or program before it is fully baked? See how they use it, and then modify to best meet their needs.

How about allowing employees to pick the shifts they want to work? Define their job responsibilities? Or create the scope of their benefits plan? Instead of forcing, ask how they would roam—*then* pave.

As a parent, instead of pushing your kids into a certain career, couldn't you let them first find their own interests? Then, once they've made "tracks in the grass," pave it with support where it is most needed?

*Paul Williams is the founder of international marketing firm Idea Sandbox. You can contact him at **paul@idea-sandbox.com** or visit **www.idea-sandbox.com**.*

A Healthy Whack

This article is based on an idea from Roger von Oech's *Creative Whack Pack*, a deck of creative strategy cards meant to "provoke and inspire your thinking."

The cards are actually chunks Roger has extracted from his book, *A Whack on the Side of the Head: How You Can be More Creative*. I highly recommend the Whack Pack and the book!

Finally, for those of you with iPhones, Roger has just released his *Creative Whack Pack* in iPhone format (you can search for it at iTunes) When you need creative inspiration, power-up your iPhone (or iTouch) and let the ideas flow. It's an excellent application!

I'm sure there are many more ways to apply this lesson. Be aware of when you may be forcing, and see if you can ease up and see what develops. MSC

*This article is posted at **www.mpdailyfix.com**.*

Phil Gyford, www.gyford.com

Have an opinion you'd like to share in "Topping Out"? Send your feedback to Geoff Weisenberger, senior editor, at weisenberger@modernsteel.com.

STRUCTURAL AND EARTHQUAKE ENGINEERING SOFTWARE

A message from Ashraf Habibullah

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